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Paper 1215 PO 2

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FLY ASH CONCRETE FOR SHIPPINGPORT ATOMIC POWER STATION

Robert J. McAllister,[1] A.M. ASCE
(Proc. Paper 1215)

ABSTRACT

About 15,000 cubic yards of fly ash concrete using type IS Portland Blast Furnace Slag Cement have been placed at the Shippingport Atomic Power Station. This paper deals with the ways in which the solution of many problems was accomplished, with particular emphasis on the development of a mix designed to attain a compressive strength of 3,000 lbs. per square inch at 28 days.

INTRODUCTION

Early in 1955, Duquesne Light Company decided to determine if fly ash could be used successfully in the concrete at the Shippingport Atomic Power Station. Here, it was estimated, about 50,000 cubic yards of concrete were required. This appeared to be an ideal job on which to obtain the advantages of improved quality of concrete along with a reduction in concrete costs that the literature on fly ash concrete indicated could be expected.

Many questions needed answers before fly ash concrete could be placed in the forms. Such things as the availability of fly ash of suitable quality in large quantities, methods of handling, transporting and batching in job size quantities, and the development of reliable job mixes were problems which had to be solved.

This paper will deal with the ways in which the solution was accomplished with particular emphasis on the development of a mix which was designed to attain a compressive strength of 3,000 lbs. per square inch at 28 days. As of July 1, 1956, about 15,000 cubic yards of 3,000 lb. fly ash concrete have been placed at the Shippingport Atomic Power Station. In this quantity of concrete about 680 tons of fly ash were used.

Note: Discussion open until September 1, 1957. Paper 1215 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 2, April, 1957.

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Preliminary Testing

A preliminary test program was set up with a two-fold objective. First it was necessary to check the quality of the fly ash and to determine the uniformity in quality which could be expected. Secondly, it was desirable to obtain basic data on fly ash concrete using the same materials that were being used on the job and, if possible, to set up a basis for the design of satisfactory job mixes.

Fly Ash Quality

The chemical and physical characteristics of fly ash are affected by the type of coal, the degree of pulverization of the coal and the efficiency of combustion. Therefore, fly ash from a particular source should be judged on its own merit as to its suitability for use in concrete. A good yardstick to determine its suitability is its compliance with ASTM Designation C350-54T, Tentative Specification for Fly Ash for Use as an Admixture in Portland Cement Concrete.

The Frank R. Phillips Power Station was chosen tentatively as the source of fly ash for Shippingport. To determine the quality and characteristics of the fly ash from this station, a schedule of sampling was set up such that four samples per day would be taken during a two week period from each of the five boilers then in the station. Since the loading conditions on a boiler have an effect on its combustion, two samples were to be taken during peak hours and two during off-peak hours. A total of 137 samples was taken.

Specific gravity, fineness, moisture content and loss on ignition were determined from each individual sample. Weekly composite samples for each boiler were subjected to a chemical analysis. A summary of the results of this testing is tabulated in Table I and compared with ASTM Specification C350-54T. It will be noted that, although some individual samples do not comply, all of the average values fall within the ASTM requirements, except in one case where the fineness is below the minimum. Boiler 5 produced some fly ash during high load conditions which showed a loss on ignition as high as 19.56%. This is an indication of high carbon content and was traced to a faulty burner. This condition was, of course, corrected as soon as possible. The lack of uniformity and the coarseness of Boiler 3 fly ash have been associated with pulverizer performance.

While these non-conformities were disturbing they were not alarming. It is believed that thorough mixing of the fly ash from all of the boilers is accomplished by the time the fly ash is transported from the collectors to the storage silo and is discharged to trucks or other conveyances. This has been substantiated by data obtained during control testing of fly ash actually used on the project. These data representing the fly ash used prior to July 1, 1956 show an average ignition loss of 4.21%, the high and low values being 5.96% and 3.12% respectively, with over 90% of the values falling between 3.5 and 5.5%. The fineness data for the same fly ash averaged 3469 square centimeters per gram (Blaine), the high and low values being 3930 and 3075, with 95% of the values falling between 3300 and 3800. The average loss on ignition of the preliminary series of tests was 4.72% while the average fineness was 3330 sq.cm./g.

Laboratory Concrete Test Mixes

The fly ash tests indicated that the fly ash from Phillips Power Station appeared to be suitable for use in concrete. Therefore, the second phase of the test program—to design and test fly ash concrete mixes—was initiated. An independent commercial testing laboratory was retained to conduct this series of tests.

The purpose of this group of tests was to determine the optimum proportion of fly ash to use and to establish a basis for the design of job mixes. The immediate objective was to set up a good 3,000 lb. fly ash concrete mix which could be used in the turbine generator portion of the Shippingport Atomic Power Station.

The cement, sand and gravel were obtained from the job site batching plant while the fly ash came from Phillips Power Station. Since type I Portland Cement has become increasingly more difficult to obtain in quantity in the Pittsburgh district, it has been virtually replaced with type IS Portland Blast Furnace Slag Cement. Accordingly, type IS cement meeting ASTM Designation C205-53T was used. All materials were subjected to the usual tests in accordance with the applicable ASTM Specifications.

Unfortunately, no published data were available on the use of fly ash with type IS cement. Since type IS cement contains blast furnace slag, which is a pozzolanic material, interground with the cement clinker, there was some concern about using fly ash, also a pozzolan, with the slag cement. However, it was decided that type IS cement should be used in these tests for two reasons. Type IS cement was being used at the job for regular concrete and would undoubtedly have to be used for fly ash concrete. Secondly, if fly ash is sold for use in concrete, prospective users in the Pittsburgh district would be faced with the same situation. It was considered essential to determine what results could be obtained from the use of fly ash with type IS cement.

The series of tests was divided into seven basic groups of five mixes each. In each group the first mix was made without fly ash and the remaining four mixes contained fly ash in the proportions of 10%, 20%, 25% and 30% of the absolute volume of the cementitious materials with the total absolute volume of the cementitious materials remaining constant. The base mixes in the first five groups were non-air entrained and contained respectively 4, 4.5, 5, 5.5 and 7 sacks of cement per cubic yard. Groups 6 and 7 were the same as groups 4 and 5 respectively, with the addition of an air entraining agent. The air entrainment was specified as 3 1/2 ± 1%.

A total of 630 standard 6" x 12" compression cylinders was made consisting of 18 cylinders for each mix. These were tested in triplicate at 1, 3, 7, 28, 60 and 90 days. All cylinders were made and tested in accordance with ASTM C192-54 and C39-49. The cardboard molds were removed at 20 to 24 hours after casting the specimens with no damage to any of the specimens due to insufficient setting or hardening of the concrete.

In virtually every case, the base mix produced the strongest cylinders in each group at all ages with a steady loss of strength as the cement content was decreased and the fly ash content increased correspondingly. Space prohibits the inclusion of all the strength data in this paper. Tables 2 and 3 show the mixes and compressive strength results for Groups 4 and 5. These two groups are of particular interest since the base mixes are approximately the same as the job mixes for 3,000 lb. and 4,000 lb. non-fly ash concrete.

From the viewpoint of compressive strength requirements, the results of these tests were not up to expectations. It was believed that the desired mix for any given strength should be a variation of the regular non-fly ash base mix for that strength. If this were so, then the fly ash mix would be economically practical and would confirm the concensus of published opinions that fly ash can be used to replace a portion of the cement without materially affecting the strength. It appeared from the lack of sufficient margin in the test results there was a question of consistently obtaining the desired design strength at 28 days from the fly ash mixes. However, it was noted that mix DL-4-1 (without fly ash) showed a 28 day strength below the results being attained by the same mix on the Shippingport project. It had been noted previously in other cases that the field control cylinders from Shippingport have consistently tested to higher values than did cylinders made from laboratory verification mixes using the same proportions and materials. This has been attributed by the laboratory to better mixing which is possible in the larger batches used on the job.

The laboratory report indicated that 300 lbs. of water were used for all mixes for a 3 to 4 inch slump. This was disturbing since previous experience had shown that fly ash mixes generally require from 1 to 3 gallons less per cubic yard than regular mixes of the same slump. This reduction, although small, may be a factor in increasing the strength of fly ash concrete.

Figure I shows a plot of pounds of cement per cubic yard against pounds of fly ash with contour lines of strength. These lines of strength were developed from the 28 day compressive strength results which were obtained in the preliminary laboratory tests just described. The curves show that a given strength can be obtained by using fly ash in amounts up to 20% by volume with corresponding reductions of the cement content. Above the 20% fly ash line the curves tend to become more vertical indicating that, as more fly ash is added to the concrete, the cement content should be held nearly constant to maintain strength. However, above the 25% line the curves begin to reverse again. It is apparent that further investigation is necessary to clearly establish the form of the curves when the fly ash content is more than 25-30% of the cementitious materials.

A study of Figure I and of the cylinder strengths led to a decision to use fly ash in the amount of about 20%, by volume, of the cementing materials. This amount appeared to be a conservative compromise and is in line with previous experience where fly ash was used with Type I cement.

Field Concrete Test Mixes

For further testing, mixes DL-4-3 and DL-5-3, basically 5 1/2 bag and 7 bag mixes with 20% fly ash, were chosen as the most promising mixes for 3,000 lb. and 4,000 lb. concrete. To duplicate field conditions as closely as possible, this concrete was batched and mixed at the batching plant serving the Shippingport project. The mixing was performed in a 3 cubic yard drum type mixer loaded to a capacity. Each of the two mixes was made to 1 1/2", 2 1/2" and 3" slumps. Only the water content was changed to vary the slump. For each slump 9 standard 6" x 12" compression cylinders were made and tested in triplicate at 7, 14 and 28 days.

The compression test values for the 4,000 lb. mix (DL-5-3) at 28 days were 3800, 3720 and 3480 lbs. per square inch for the 1 1/2", 2 1/2" and 3" slumps, respectively. These results were puzzling and were not readily ex-

plained since this was the same mix which had produced a three cylinder average of 4170 lbs. per sq. in. for 3 to 4 inch slump in the previous series of tests. According to the laboratory these should be assumed as "freak" values pending additional testing.

The 3,000 lb. mix (DL-4-3), on the other hand, appeared to be satisfactory. Figure II shows the compressive strength plotted against slump for this mix.

Both mixes appeared to be of good quality and exhibited a marked degree of cohesion between the aggregates and the paste without being sticky. This concrete was workable and, even with 1 1/2" slump, flowed when subjected to a little agitation.

On the basis of these results, a decision was made to use the 3,000 lb. fly ash concrete in the turbine-generator portion of the new station. A maximum slump of 3 inches was tentatively specified.

Job Concrete

Concrete Mixes

On October 31, 1955, the first 3,000 lb. fly ash concrete was placed in the turbine-generator portion of the Shippingport Atomic Power Station. After a study of the field results obtained in placing 3,000 lb. fly ash concrete, the Atomic Energy Commission sanctioned the use of this concrete in the nuclear portion of the plant. Accordingly, on January 6, 1956, the first fly ash concrete was placed in the nuclear area. The intervening ten weeks provided an excellent opportunity to evaluate fly ash concrete under field conditions. During that period, both fly ash and regular 3,000 lb. concrete were being made of the same materials and were being placed concurrently under similar conditions.

Figure III shows comparative average 28 day compressive strengths for the two mixes at weekly intervals over the ten week period. Also shown are the quantities of each type of concrete placed during each week and the design mix for each type. A total of about 7,500 cubic yards of fly ash concrete and about 6900 cubic yards of non-fly ash concrete were placed in this period. The overall average compressive strengths for the 3,000 lb. fly ash concrete at ages 7, 14 and 28 days were 2218, 3037 and 3954 lbs. per sq. in., respectively; for the 3,000 lb. regular concrete, 2548, 3257 and 4048 lbs. per sq. in., respectively. These results are plotted on Figure IV and here it can be seen that, although the regular concrete shows a greater strength at each age, the margin has been narrowed with increasing age. It appears that the compressive strength of the fly ash concrete may, at a later age, exceed that of the regular concrete. This has been reported in the literature to be the case when fly ash is used with type I cement. During the operations at Shippingport, additional cylinders were made from a few random pours for testing at 45 and 90 days. Figure V shows the plot of the results of testing these cylinders. The fly ash concrete curve is the average of ten groups of cylinders while the regular concrete curve is the average of nine groups. These curves, while not backed by as extensive data as the two previous sets of curves, tend to indicate that concrete made with fly ash reaches greater strengths at later ages than non-fly ash concrete when type IS cement is used as it does when type I cement is used.

In addition to the 3,000 lb. fly ash and non-fly ash concrete, the following strengths and types of concrete have been used on the Shippingport project:

4,000 lb. non-fly ash, 3,000 air entraining with and without fly ash.

All of the 4,000 lb. concrete required was placed in the early stages of construction prior to the completion of the laboratory tests on fly ash concrete. Therefore, there was no pressing need to develop a satisfactory 4,000 lb. fly ash mix and consequently no additional work has been done on the higher strength mix. However, there appears to be little doubt that fly ash can be used in 4,000 lb. concrete with the same satisfactory results as have been obtained with its use in 3,000 lb. concrete at the Shippingport Atomic Power Station.

To date only a relatively small quantity of 3,000 lb. air entraining fly ash concrete has been placed. This mix was developed from laboratory verification mixes in addition to the initial series of tests discussed previously. The proportions of this mix are 462 lbs. of type IS cement and 102 lbs. of fly ash per cubic yard with a water cement ratio of 0.53. Five ounces of Darex per sack of cement are required to obtain the specified air content of 3 1/2 + 1%. This compares to about 1.5 ounces per sack for mixes without fly ash.

The compressive strengths attained by this mix have been satisfactory. A series of tests is currently underway to test fly ash air entraining concrete as to permeability and resistance to frost action along with comparative mixes. This testing has not progressed sufficiently nor has there been enough fly ash air entraining concrete placed in this job to properly evaluate this type of concrete at this time.

Field Experience

No great difficulties have been encountered in handling this type of concrete on the job. The concrete with 2 to 3 inch slump can be placed in the forms as readily with the use of either chutes or elephant trunks as regular concrete with a 3 to 4 inch slump. Discharge of the concrete with 1 1/2 inch slump from the high discharge type of mixer trucks is somewhat slow and the concrete moves sluggishly in the chutes. However, it is believed that fly ash concrete is no worse in this respect than any other concrete of stiff proportions. Once in the forms, the fly ash concrete, including that with 1 1/2 inch slump, can be moved and worked around reinforcing steel readily when vibrated. Because of the cohesiveness of the fly ash concrete segregation of the aggregates has been at a minimum.

It was noted that fly ash concrete gained its strength more slowly than concrete made without fly ash. This was more noticeable during cold weather even though the same curing procedure was used for concrete made with or without fly ash. Under some conditions this may cause some delay in the removal of the forms, with the delay being more pronounced in cold weather than in warm weather.

Due to its buttery consistency, fly ash concrete handles somewhat differently during finishing than regular concrete. However, in a short time the finishers became accustomed to working with it and they have been able to achieve excellent results with a minimum of trowelling and little or no bleeding. Furthermore, the concrete, although cohesive, does not stick to the finishing tools. Formed surfaces obtain a dense smooth surface remarkably free from voids and honeycombing.

Fly ash concrete, in both the fresh and hardened states, is a few shades darker than non-fly ash concrete. However, the change in color is not so great that it makes the fly ash concrete objectionable from an appearnace standpoint.

Quality Control

Fly Ash

A constant check has been maintained on the quality of the fly ash furnished to the job. Through November and December, 1955, a tube sample was taken from each truckload before it left Phillips Power Station. These samples were checked for loss on ignition, moisture content, and fineness. At the same time one sample of fly ash representative of the fly ash which went into each 100 cubic yards of concrete was taken at the batching plant by a representative of a commercial laboratory. These samples were tested by the laboratory for loss on ignition, moisture content and fineness. A composite sample was made from a maximum of each five consecutive batching plant samples. A minimum of one composite sample was made for each pour. The composite samples were analyzed for chemical composition. All sampling and testing was done essentially in accordance with ASTM Specification C311-54T.

It was noted from the above procedure that the results of testing the truck samples corresponded very closely to those obtained in testing the batching plant samples. Hence, the conclusion was reached that the fly ash from the various boilers was completely mixed in its passage from the collectors to the storage silo and then to the trucks. Therefore, the sampling and testing at the source was discontinued. It was believed that testing the fly ash at the batching plant was preferable since this gives closer association between the fly ash test results and the concrete pours.

The testing procedure at the batching plant was continued as described until February, 1956. Because the tests showed that the fly ash had good uniformity and good compliance with the ASTM Specification C350-54T, the amount of testing was reduced. Since that time one sample representing each 100 cubic yards of concrete has been taken from the storage bin and five such consecutive samples combined into a composite sample. Testing of the composite sample has consisted of determination of fineness and specific gravity along with the chemical analysis as specified in ASTM C-350.

Other Materials

Sand, gravel and cement have been checked periodically for compliance with the applicable ASTM Specifications such as is done on any large job on which controlled concrete is used.

Handling Fly Ash

A silo of 1200 ton capacity was installed at Phillips Power Station so that the fly ash from all of the boilers could be handled dry. Prior to this time the fly ash had been pumped to settling basins from which it was later removed for disposal. The silo was equipped with air slides and gate valves to permit loading the fly ash into trucks or railroad cars.

Transportation from Phillips Power Station to Shippingport is accomplished by the use of a specially converted tandem dump truck with a capacity of 14 tons of fly ash. This truck was provided with a dust-tight cover through which are openings for filling and venting. Loading of the truck is carried out by connecting the silo outlet to the truck filler with a canvas boot and similarly connecting the truck vent opening to a breather pipe which projects

through the bottom of the silo and extends upward so that its opening is near the inside top of the silo. The purpose of the breather pipe is to carry off the air and fly ash which are forced out of the truck and thus provide relatively dust free loading.

Dust-tight and watertight storage and handling facilities were also required at the batching plant. This was achieved by using a screw conveyor with a flanged hopper and a bucket elevator to transport the fly ash from the truck to a 1200 barrel storage bin. All of this equipment is of the same type as is used to handle cement. A canvas boot with steel flanges on each end is used to make the connection between the truck discharge opening, which is in the tailgate, and the conveyor hopper. It has been found necessary to attach a small electrically operated vibrator to the body of the truck to start and keep the fly ash moving from the truck into the hopper.

Cost

The use of fly ash in the 3,000 lb. concrete at Shippingport Atomic Power Station has resulted in a saving of about 60 cents per cubic yard. This represents a reduction of between 4 and 4 1/2% in the cost of concrete delivered to the job site.

TABLE I

SUMMARY

CHEMICAL AND PHYSICAL CHARACTERISTICS OF FLY ASH FROM BOILERS NO. 1-5, PHILLIPS POWER STATION

	ASTM Limits	Boiler 1	2	3	4	5
Moisture, % by wt.	3.0 Max.	-	-	-	-	-
Avg.		0.17	0.15	0.11	0.15	0.24
Max.		.38	.28	.26	.36	.28
Min.		.01	.04	.02	.02	.12
Samples - number tested		36	11	36	36	18
- above max.		0	0	0	0	0
Loss on Ignition (750°C)	12.0 Max.	-	-	-	-	-
Avg.		4.86	5.05	3.96	3.30	8.50
Max.		11.10	9.88	6.04	6.38	19.56
Min.		1.89	3.28	2.03	1.54	3.27
Samples - number tested		36	11	36	36	18
- above max.		0	0	0	0	5
Silica, SiO_2 (dry)	40.0 Min.	48.6	41.1	46.8	48.6	45.9
Magnesium oxide, MgO, % by wt. (dry)	3.0 Max.	0.8	0.7	0.7	0.6	0.5
Sulfur trioxide, SO_3, % by wt. (dry)	3.0 Max.	0.4	0.5	0.5	0.4	0.4
Available alkalies as Na_2O, % by wt.	1.5 Max.	0.5	0.6	0.9	0.9	1.5
Specific Gravity, g/cm^3	± 5% of Avg.	-	-	-	-	-
Avg.		2.39	2.34	2.37	2.44	2.35
Max.		2.46	2.38	2.43	2.49	2.43
Min.		2.31	2.18	2.31	2.39	2.23
Samples - number tested		36	11	36	36	18
- Outside uniformity limits		0	1	0	0	0
Fineness, sq. cm/g.	(2800 Min.					
Avg.	(±15% of Avg.	3730	3248	2593	3558	3598
Max.		4718	3560	3743	4638	4092
Min.		2969	2223	1482	2640	2937
Samples -						
Number tested		36	11	36	36	18
Below Min.		0	1	18	1	0
Outside Uniformity Limits		4	1	24	8	2

Loss on Ignition - Overall Average . . . 4.72%
Fineness - Overall Average . . . . . 3330 Sq. cm/g

TABLE 2

COMPARATIVE STRENGTHS OF NOMINAL 5.5 BAG MIXES

PRELIMINARY TEST SERIES

Mix.	Cement lb. per Cu. Yd.	Fly Ash lb. per Cu. Yd.	Compressive Strength p.s.i. 1 Day	3 Days	7 Days	28 Days	60 Days	90 Days
4-1	517.0	--	750	1485	2510	3470	3880	4100
4-2	465.3	46.0	698	1330	2210	3130	3760	3890
4-3	413.6	91.0	523	1130	1790	3040	3410	3640
4-4	387.8	116.0	466	1025	1640	2820	3320	3505
4-5	361.8	138.0	365	955	1520	2730	3210	3400

1170 lbs. sand, 2020 lbs. gravel (1½" max.), and 300 lbs. water used in each above mixes.

TABLE 3

COMPARATIVE COMPRESSIVE STRENGTHS OF NOMINAL 7 BAG MIXES

PRELIMINARY TEST SERIES

Mix.	Cement lb. per Cu. Yd.	Fly Ash lb. per Cu. Yd.	Compressive Strength p.s.i. 1 Day	3 Days	7 Days	28 Days	60 Days	90 Days
5-1	658.0	--	1155	2285	3290	4500	4600	4880
5-2	592.5	58.5	1005	1945	3005	4250	4450	4810
5-3	526.4	117.0	750	1700	2675	4170	4330	4700
5-4	494.5	145.0	675	1520	2265	3960	4210	4460
5-5	460.6	175.0	635	1450	2120	3870	4000	4170

1150 lbs. sand, 1860 lbs. gravel (1½" max.) and 300 lbs. water used in each of above mixes.

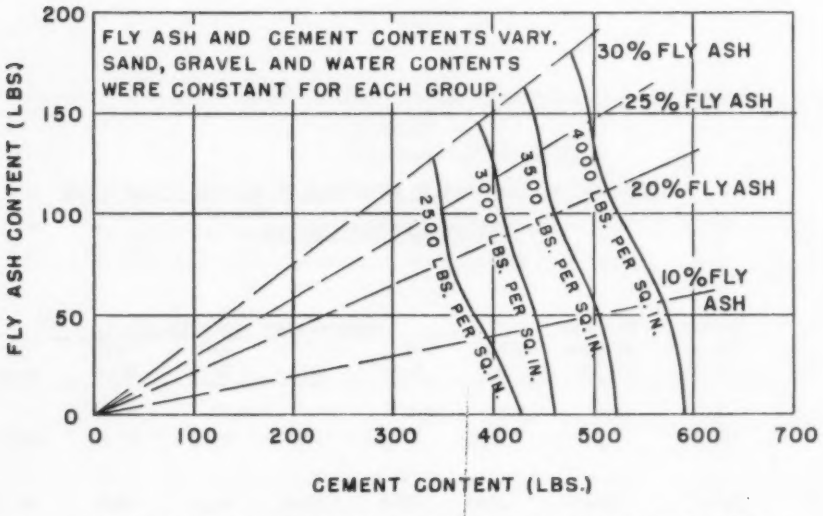


FIG. I—28 DAY COMPRESSIVE STRENGTH CONTOURS FOR NON AIR ENTRAINED MIXES—LABORATORY TESTS

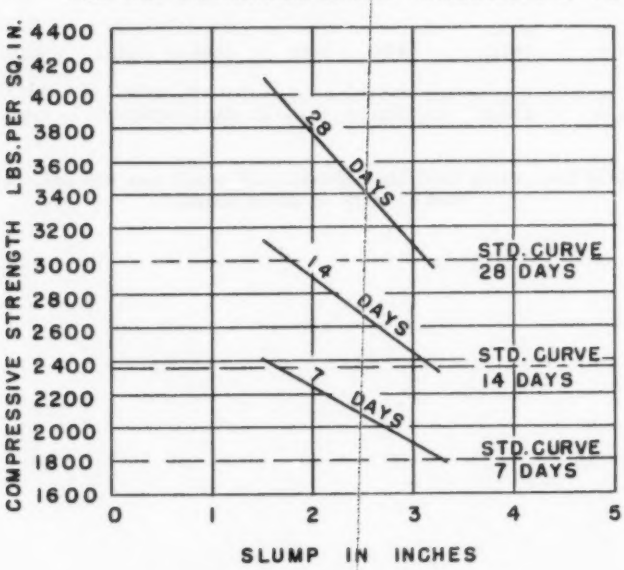
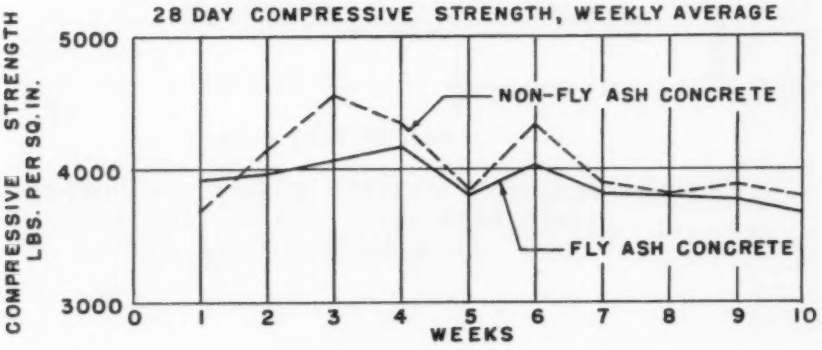


FIG. II—COMPRESSIVE STRENGTH VS SLUMP FOR 3000 LB. FLY ASH CONCRETE-FIELD TEST

APPROX. QUANTITIES OF CONCRETE PLACED, CUBIC YARDS.

WEEK	1	2	3	4	5	6	7	8	9	10
FLY ASH	700	700	1600	300	1100	2000	100	400	400	300
ION-FLY ASH	100	400	400	700	800	1100	1600	800	500	500



MATERIALS PER CUBIC YARD
3000 LB. CONCRETE

CEMENT LBS.	FLY ASH LBS.	WATER GALS.	SAND LBS.	GRAVEL LBS.
414	91	33	1138	2052
517	—	33	1125	2070

FIG. III — COMPARISON OF FLY ASH AND NON-FLY ASH CONCRETE PLACED FROM OCT. 31, 1955 TO JAN. 6, 1956

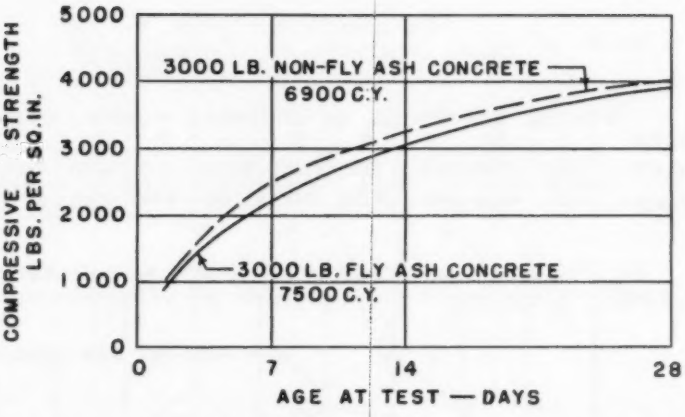


FIG IV—COMPRESSIVE STRENGTH CURVES FOR CONCRETE PLACED FROM OCT. 31, 1955 TO JAN 6, 1956 FIELD CONTROL TESTS

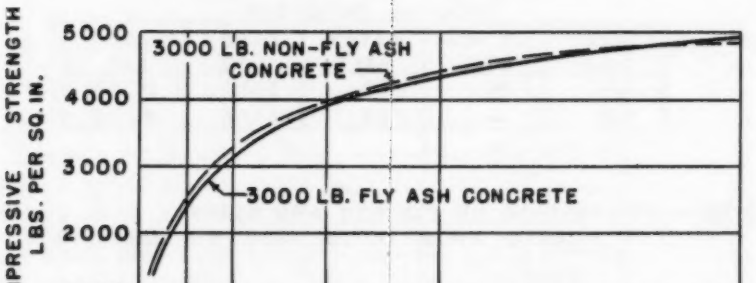


FIG V—COMPRESSIVE STRENGTH AT LATER AGES FIELD CONTROL TESTS

Paper 1216 PO 2

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

WATER-HAMMER DESIGN CRITERIA

John Parmakian,[1] M. ASCE
(Proc. Paper 1216)

SYNOPSIS

A design criteria for including the effects of water-hammer in turbine penstock and pump discharge line installations is outlined. Various possible conditions of operation are enumerated and suitable factors of safety are assigned to each contingency. Typical examples are included.

INTRODUCTION

The method of computing water-hammer in turbine penstock and pump discharge line systems for most of the conventional types of installations is well established. After the physical and flow characteristics of the installation are defined and the operating conditions are established, it is usually not too difficult to determine the pressure surges throughout the system with a satisfactory degree of accuracy for design purposes. In many instances the designer is faced with the necessity of determining the design pressure requirements for the penstock and its turbines or the pump discharge line and its pumps long before the turbine or pump contract has been awarded. As a result, the actual characteristics of the turbines, pumps, or control equipment which are to be used may not be known at the time that the water-hammer allowances must be established for the conduits. Fortunately, these characteristics can be estimated in advance for most hydraulic equipment to permit a timely satisfactory allowance for water-hammer for the system.

There are many possible operating conditions capable of producing pressure surges at power and pumping plants. If practical, it would be desirable to design all component parts of an installation for the most severe water-hammer condition possible with liberal factors of safety. However, this will usually be found to be uneconomical. Hence, the designer is faced with the

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practical choice of considering the probability of the various possible contingencies and evaluating suitable factors of safety for each.

In order to assist the designers in the Division of Design at the Bureau of Reclamation, the following water-hammer design criteria has been proposed. In addition to providing a standard or basis for design, the water-hammer criteria insures that in those cases where alternate designs of different materials are prepared, the designs can be evaluated on a comparable basis. The contingencies in this criteria have been divided into three conditions of operation with suitable factors of safety assigned to each. These conditions of operation are as follows:

Normal conditions of operation,
Emergency conditions of operation,
Emergency conditions of operation not considered as a basis for design.

Normal Conditions of Turbine Operation

The turbine penstock installation may be operated at any head between the maximum and minimum values of the forebay water surface elevation.

The turbine gates may be moved at any rate of speed by the action of the governor head up to a predetermined rate, or at a slower rate by manual control through the auxiliary relay valve.

The turbines may be operating at any gate position and be required to drop any or all of the load.

If the turbine penstock installation is equipped with any of the following pressure control devices, it will be assumed that these devices are properly adjusted and function in the manner for which the equipment is designed:

a) Surge tanks
b) Relief valves
c) Governor control apparatus
d) Cushioning stroke device
e) Any other pressure control device

Unless the actual turbine characteristics are known, the effective flow area through the turbine gates during the maximum rate of gate movement will be taken as a linear relation with respect to time.

The water-hammer effects will be computed on the basis of governor head action for the governor rate which is actually set by stops in the governor for speed regulation. If the relay valve stops are adjusted to give a slower governor setting than that for which the governor is designed, this rate shall be determined prior to proceeding with the design of turbine penstock installation and later adhered to at the powerplant so that an economical basis for designing the penstock, scroll case, etc., under normal operating conditions can be established. In those instances where due to a higher reservoir elevation it is necessary to set the stops on the main relay valve for a slower rate of gate movement, the water-hammer effects will be determined for this slower rate of gate movement also.

The reduction in head at various points along the penstock will be computed for the maximum required load addition and for the rate of gate opening which is actually set by stops in the governor in those cases where it appears that the profile of the penstock is unfavorable. This minimum pressure will then be used as a basis for normal design of the penstock to insure that water

column separation does not occur and that subatmospheric pressures will not cause a penstock failure due to collapse.

A surge tank shall be provided where the resulting reduction in water-hammer pressures will provide a more economical turbine penstock installation. A surge tank shall also be provided if the generating unit is required to regulate frequency for the system and the speed rise based upon the rejection of the entire rated load for a unit operating independently cannot be reduced below 45 percent by other practical methods. The speed rise parameter for full load rejection is used here solely as an index of the regulating ability of the unit.

If a surge tank is present in the penstock system, the upsurge in the surge tank will be computed for the maximum reservoir level and the rejection of the turbine flow corresponding to the rated output of the generator during the gate traversing time which is actually set by stops in the governor. Unless an overflow spillway is provided, the top elevation of the surge tank will be determined by adding a suitable freeboard to the maximum height of water at the highest upsurge.

The downsurge in the surge tank will be computed for the minimum reservoir level condition for the maximum required load addition and for the gate traversing time which is actually set by stops on the governor. The bottom of the surge tank will be located at a sufficient distance below the lowest downsurge in the tank to safeguard against air entering the penstock.

The turbine, penstock, surge tank and other pressure control devices will be designed to withstand the conditions of normal operation which are given above with a minimum factor of safety of 4 based on the ultimate bursting strength and a suitable factor of safety against collapse.

Normal Conditions of Pump Operation

Pumps may be started or tripped manually or automatically throughout the entire range of pumping heads specified for the installation.

If shutoff valves are present in the discharge line, the pump casing and portions of the pump discharge line between the pump and the valve will always be subjected to the shutoff head of the pump.

On long pump discharge lines which are initially full, a considerable length of the discharge line beyond the quick opening shutoff valves may be subjected to practically full shutoff head upon the rapid starting of the pumps.

Power failure at all pump motors in the pumping plant will be assumed to occur a sufficient number of times during the operation of the pumping plant to be considered a normal operating condition.

If the pump discharge system is equipped with any of the following pressure control devices, it will be assumed that these devices are properly designed and adjusted, and function in the manner for which the equipment is designed:

a) Surge tanks
b) Air chambers
c) Check valves
d) Surge suppressors
e) Other pressure control devices

The minimum pressure gradient along the length of the discharge line due to power failure will be investigated to insure that water column separation

does not occur. This minimum gradient will be used as a basis for the normal design of the pump discharge line to insure against failure due to collapse. If the computed minimum pressure gradient at any point in the discharge line falls below the vapor pressure of water, means will be taken to avoid the condition of water column separation. If this is not practical, special devices may be used to minimize the pressure rise created by the subsequent rejoining of the water columns.

If a check valve is present in the discharge line, the water-hammer effects will be computed on the basis that the check valve closes immediately upon the reversal of flow.

If a surge tank is used, the top of the tank will be determined by adding a sufficient freeboard to the highest upsurge attained in the surge tank to prevent spilling unless an overflow spillway is provided. The bottom of the surge tank will be located at a sufficient distance below the lowest downsurge to safeguard against air entering the discharge line.

If an air chamber is present in the pump discharge line system, the water-hammer effects will be determined by using the minimum volume of air in the air chamber at which point the air compressor would normally start up.

The head for which the pump and discharge line will be designed will be taken to include the water-hammer effects for the conditions of normal operation listed above with a minimum factor of safety of 4 based on the ultimate bursting strength of the structure and a suitable factor of safety against collapse.

Emergency Conditions of Turbine Operation

The turbine gates may be closed at any time by the action of the governor head, manual control knob with the main relay valve or the emergency solenoid device.

The cushioning stroke will be assumed to be inoperative on one unit.

If a relief valve is present, it will be assumed to be inoperative on one unit.

The gate traversing time will be taken as the minimum time for which the governor is designed.

The water-hammer at the turbine and along the length of the penstock will be computed for the maximum reservoir head condition for final part-gate closure to the zero-gate position on one unit at the maximum governor rate in $\frac{2L}{a}$ seconds.

If a surge tank is present in the penstock system, the upsurge in the surge tank will be computed for the maximum reservoir head condition for the rejection of full gate turbine flow of all units at the maximum rate for which the governor is designed. In determining the top elevation of the surge tank, nothing will be added to the upsurge for this emergency condition of operation.

The turbine, penstock, surge tank, relief valve or other control devices will be designed to withstand the above emergency conditions of operation with a minimum factor of safety of 2 based on the ultimate bursting or collapsing strength.

Emergency Conditions of Pump Operation

The emergency conditions for electrically driven centrifugal pumps are

assumed in general to be caused by malfunctioning of the pressure control equipment as follows:

If surge suppressors or pressure relief valves are present, one will be assumed to be inoperative.

If a check valve is used to shut off the return flow through the pump, the check valve closure of one unit will be assumed to be delayed and to occur at the time of maximum reverse flow.

If air inlet valves are present, they will be assumed to be inoperative.

If an air chamber is present in the pump discharge line system, the water-hammer effects will be determined by using the minimum volume of air in the air chamber at which point an emergency tripout of all pumps on the discharge line will occur.

The head for which the pump and discharge line will be designed will be taken to include the water-hammer effects for the conditions of emergency operation listed above with a minimum factor of safety of 2 based on the ultimate bursting or collapsing strength of the member.

Emergency Conditions of Turbine Operation Not Considered for Design

Other possible emergency conditions of turbine operation are those during which certain pieces of control equipment malfunction in the most unfavorable manner. The most severe emergency condition of operation which will yield the maximum head rise in a turbine penstock installation will be caused by a very rapid closure of the turbine gates when the flow of water in the penstock and the head is a maximum. The maximum head rise in feet due to this condition of operation is about 100 to 125 times the water velocity in feet per second.

Since such conditions of operation require an abnormal malfunctioning of all control equipment at the most unfavorable moment, the probability of obtaining this type of operation is considered exceedingly remote. Hence, these conditions are not ordinarily used as a basis for design. However, it is desirable that the stresses in the various parts of the structure are not in excess of the ultimate bursting strength for these emergency conditions of operation.

Emergency Conditions of Pump Operation Not Considered for Design

These emergency conditions of pump operation are those during which various equipment malfunctions in the most unfavorable manner, such as a pump shaft failure, instantaneous seal ring seizure, loss of all air in the air chamber, very rapid abnormal closure of a gate or valve, etc. Since these conditions of operation require an abnormal malfunctioning of various apparatus, the probability of obtaining this type of operation is exceedingly remote. However, after the design has been established from other considerations, it is desirable that the computed maximum pressure in the various parts of the structure are below the ultimate bursting strength.

Application for a Turbine Penstock

An application of the above criteria for determining the water-hammer allowances at a turbine penstock installation will now be given. Consider the Bureau of Reclamation's Boysen Powerplant in Wyoming. The powerplant has the following characteristics:

Number of turbines, 2

Turbine rating, 10,500 horsepower at 96-foot net head

Generator rating, 8,333 kva, Power factor 0.90

Penstock length, 635 feet

Penstock diameter, 15 feet

Maximum gross head on turbine, 124 feet

Minimum gross head on turbine, 70 feet

Full gate discharge at 121 feet net head, 1,355 cubic feet per second per unit

Full gate discharge at 96 feet net head, 1,190 cubic feet per second per unit

Full gate discharge at 67 feet net head, 995 cubic feet per second per unit

Round trip water-hammer wave travel time from turbine to reservoir, 0.35 seconds.

At this installation the reservoir and tail water elevations undergo large variations and the gross head on the units varies from 70 to 124 feet. The rated capacity of the turbine and generator is attained when the turbine is operating at the full gate position with a net effective head of 96 feet. The turbine governors are designed and are set in the field so that they can close or open the turbine wicket gates from the full gate position to the speed-no-load position in a minimum time of 5 seconds. The sevomotors also have a cushioning stroke which starts to operate during the closing stroke of the turbine gates at the 10 percent gate position. The speed-no-load position is at about 12 percent gate position. The flow characteristics through the turbine wicket gates is assumed to be unknown and is taken as a straight line function with respect to time.

In determining the water-hammer effects for design purposes for this installation, the following were considered as normal conditions of operation:

a) The cushioning stroke was assumed to be operative at both units.

b) At the rated head of 96 feet and with the maximum rate of gate travel as defined above, the total head including water-hammer was computed for full and various part gate closures on both units.

c) For a gross head of 124 feet and with the maximum rate of gate travel, the total head including water-hammer was computed for various part gate closures of both units. The maximum gate opening at the maximum head was taken as the part gate position which corresponded to the rated capacity of the turbine and generator.

d) For a gross head of 70 feet and with the maximum rate of gate opening

of both units in 5 seconds, the total head including water hammer, was computed for the condition where the turbine gates on both units are initially at the speed-no-load position.

e) The turbine scroll case and penstock were designed to withstand the above normal conditions of operation with a minimum factor of safety of 4 based on the ultimate bursting or collapsing strength.

The emergency conditions of operation which were considered in the design for this installation are as follows:

f) The cushioning stroke on the governor sevomotor of one unit was assumed to be inoperative.

g) For a gross head of 124 feet and at the maximum rate of gate travel, the total head including water hammer was computed for full and various partial gate closures of both units. At the full gate position this will result in a substantial overloading of the turbines and generators.

h) The turbine scroll case and penstock were designed to withstand the above emergency conditions of operation with a minimum factor of safety of 2 based on the ultimate bursting or collapsing strength.

The computed maximum head of the turbines including water-hammer for the conditions listed above was as follows:

For normal conditions of operation, 183 feet

For emergency conditions of operation, 251 feet

Since a minimum factor of safety of 4 was assigned for the normal conditions of operation and a factor of safety of 2 assigned to the emergency conditions of operation, the actual designs at this installation were controlled by the normal conditions of operation.

At the Boysen Powerplant an alternate design utilizing a surge tank at the powerplant was also considered but was subsequently discarded.

In utilizing the above criteria, the results of the water-hammer studies with a surge tank were as follows:

Diameter of surge tank, 30 feet

Type of surge tank, restricted orifice

Riser and restricted orifice head loss, 20 feet for a flow of 2,710 cubic feet per second into surge tank

Computed maximum downsurge from the minimum head condition upon opening both turbine gates, 15 feet

Computed maximum upsurge from the maximum head condition upon closing both turbine gates, 18 feet

Application for Pump Discharge Line

An application of the above criteria for determining the water-hammer allowances in a pump discharge line will now be given. Consider the Bureau of Reclamation's Yakima Project, Roza Division Pumping Plant Area 17 in Washington State. This pumping plant has the following characteristics:

Number of pumps, 2

Pump motor rating, 250 horsepower for each pump motor

Rated pump capacity, 6.1 cubic feet per second for each pump at 235-foot dynamic head

Discharge line length, 2,750 feet

Discharge line plate steel 22 inches outside diameter, 3/16 inches thick

Pump speed, 1,760 rpm

Pump efficiency, 85 percent

WR^2 of rotating parts and entrained water, 237 pound $feet^2$

Round trip wave travel time in discharge line, 1.74 seconds

Control valves, none at each pump

Manual or automatic restarting of individual pumps

In determining the water-hammer effects for design purposes for this installation, the following were considered as normal conditions of operation:

a) One pump may be started at a time either manually or automatically.

b) Power failure at both pump motors will occur a sufficient number of times during the operation of the pumping plant to be considered a normal operating condition.

c) The pumps and discharge line were designed to withstand the above normal conditions of operation with a minimum factor of safety of 4 based on the ultimate bursting or collapsing strength.

In view of the absence of all control valves at this installation, the emergency conditions of operation noted in the water-hammer criteria above do not apply.

For the normal conditions listed above, the computed maximum head at the center line of the discharge pipe adjacent to the pumps was 336 feet.

Paper 1217 PO 2

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: REVIEW OF EXPERIENCE

Robert E. Glover,[1] M. ASCE
(Proc. Paper 1217)

SYNOPSIS

The review presents records of the performance of some of the older arch dams to learn how well they had served the purposes of their designers. The records are presented in tabular form, supplemented by notes and pictures.

FOREWORD

This paper is one of a group presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is not known exactly how many papers will be printed from the Symposium. So far, sixteen papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dams," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity

Note: Discussion open until September 1, 1957. Paper 1217 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 2, April, 1957.

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Abutments and Cut-offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); "Arch Dams: Observed Behavior of Several Italian Arch Dams," by Dino Tonini (Proc. Paper 1134); "Arch Dams: Measurements and Studies of Behavior of Kamishiiba Dam," by H. Kimishima and C. C. Bonin (Proc. Paper 1182); "Arch Dams: Construction of the Kamishiiba Arch Dam," by K. M. Mathisen and C. C. Bonin (Proc. Paper 1183); and "Arch Dams: Review of Experience," by Robert E. Glover (Proc. Paper 1217).

Records

A Review of Experience with the older Arch Dams was made a special feature of the Symposium on Arch Dams which was held at Knoxville, Tennessee during the week of June 4-8, 1956. It was the purpose of this review to present records of the performance of some of the older Arch Dams to learn how well they had served the purposes of their designers and to profit from these experiences to the end that new designs might be improved. Records were contributed generously for this review by Engineers from many countries. Due to the kindly assistance received from the New Zealand Institution of Engineers and from the Institution of Engineers of Australia, the New Zealand, Australia, and Tasmania area is exceptionally well represented. This is fortunate since some of the longest records of all came from there.

As might be expected, there are many items of interest and value in these records. Among them are the records of the venerable dams from Italy and France. The experience with the "peripheral joint" in Italy. The records of two dams in Germany which were bombed out during World War II, records of the use of a curtain wall to protect thin dams from damage by frost as reported from Norway, experiences with alkali-aggregate expansion from the United States, records of arch dams with overflow crests from many countries, the records for two arch dams which are alleged to have failed, records of methods of design, of resistance to earthquake and many other items.

Presentation of Data

In order to present the many records in a concise and usable way as many items as possible have been tabulated. These are supplemented by extracts from the notes accompanying the records. The basic order of presentation is a geographical one which begins with Italy and ends with Tasmania. Each dam has been given a number in the table and this number appears also on the notes and on the pictures of the dams.

Summary

With records coming from so many countries it should be expected that there would be some language difficulties. It is not surprising therefore that Engineers in many cases interpreted the request for an experience record as a request for data on measurements. Although a number of dams, for this

reason, lack a specific statement covering their behavior over the years it seems certain that they have served satisfactorily, since black and white photographs were sent in nearly all cases and these were commonly supplemented with recently prepared color slides for use during the Symposium meetings. One can not study these records without being impressed with the excellent performance of Arch Dams. All types designed by all methods have served well. In three cases they have continued to stand even after an abutment or a part of the foundation was lost. By so doing they have given a convincing demonstration of the ability of the arch type of dam to cope with adverse conditions.

The original records have been placed on file in the Society Library, where they may be consulted for additional details, if desired.

NOTES

1. Ponte Alto- Ownership—Italian Government
2. Madruzza

For description see Paper 1017 by Carlo Semenza. "During the big flood of 1882, the dam was overflowed by a 5 m head of water; it suffered some damages, especially at the toe. For this reason, in 1884-1886, another dam was built 180 m downstream; the arch-dam of Madruzza, 40 m high, built of big concrete elements."

3. Osiglietta - Accizierie E Ferriere Lombarde Falck-Milan, Italy
4. Rochetta
5. Giaredo
6. Ganda
7. Publino
8. Venina
9. Valla
10. Moledana

Reported by—Dr. Ing. Mario Scalabrini—Milano, Italy. (Director of Iron and Steel Company Falck—Hydroelectric Division) Corso Matteotti 6-Milano. 6 February 1956.

11. Corfino Ownership—Societa Elettrica Selt-Valdarno—Milano, Italy
12. Turrite Di Gallicano

An earthquake in 1920 completely destroyed the village of Villa-Collemandina but did not damage the Corfino Dam.

Stability of the Turrute Di Gallicano Dam has been excellent and no repairs have been necessary. The structure withstood several earthquakes, the most severe in 1920.

These reports compiled from the 1952 records of Associazione Nazionale Imprese Distributrici di Energia Elettrica—(ANIDEL).—by Mr. R. A. Sutherland.

13. Zola—Maintained and operated by the "Service hydraulique des Bouches du Rhone—France.

The Zola Dam is a masonry arch dam whose construction was commenced in 1850 and completed in 1854. The dam is constructed of selected dressed stone with the upstream face and downstream face carefully jointed, the

RECORDS OF EXPERIENCE

Number	1	2	3
Name	Ponte-Alto	Madruzza	Osiglietta
Location	Italy	Italy	Italy
Height	46.3 M	40.6 M	76.8 M
Top Thickness	4.0 M	3.2 M	5.8 M
Base Thickness	2.0 M	5.5 M	10.74 M
Crest Length	12.0 M	15.0 M	224.0 M
Completion Date	1611-1887	1886	1939
Temperature control during construction	.....	.....	.....
Final Closure	.....	.....	Grouting
Spillway	Overflow Crest	Overflow Crest	Overflow Crest and Spillway on L. Bank.
Abutment Rock	Limestone Red Scale	Limestone Red Scale	Porphiritic Gneiss
Method of Design	.....	.....	Elastic Arch and Models.
Purpose	.....	.....	Power
Ownership	Italian Government	Italian Government	Falck
Remarks	.....	.....	Dome Type with Cushion.

RECORDS OF EXPERIENCE

Number	4	5	6
Name	Rochetta	Giardo	Ganda
Location	Italy	Italy	Italy
Height	76.0 M	27.5 M	30.0 M
Top Thickness	3.5 M	3.0 M	3.0 M
Base Thickness	12.8 M	2.0 M	3.15 M
Crest Length	136.3 M	37.43 M	90.55 M
Completion Date	1937	1941	1947
Temperature control during construction	.....	.....	.....
Final Closure	Grouting	Grouting	Grouting
Spillway	2-6x5 M Sluices	Overflow Crest	Overflow Crest
Abutment Rock	Siliceous Sandstone	Old Red Sandstone	Phyllite
Method of Design	Elastic Arch and Models.	Elastic Arch	Elastic Arch
Purpose	Power	Power	Power
Ownership	Falck	Falck	Falck
Remarks	Single Curvature Arch.	Single Curvature with Cushion.	Single Curvature with Cushion.

3 OSIGLIETTA

4 ROCCHETTA

5 GIAREDO

6 GANDA

7 PUBLINO

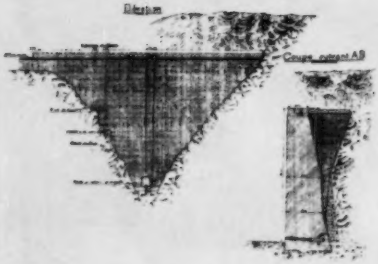
8 VENINA

RECORDS OF EXPERIENCE

Number	7	8	9
Name	Publino	Venina Lake	Valla
Location	Italy	Italy	Italy
Height	42.0 M	49.5 M	47.0 M
Top Thickness	2.8 M	0.95 M Cent. Arch 0.60 M Side Arch	1.0 M
Base Thickness	12.1 M	3.50 M Cent. Arch 1.50 M Side Arch	15.72 M
Crest Length	205.59 M	175.0 M	113.80 M
Completion Date	1951	1926	1925
Temperature control during construction	.....	.....	.....
Final Closure	Grouting	.....	None
Spillway	Glory Hole	Overflow Sill with Gates.	Overflow Crest
Abutment Rock	Gneiss	Quartziferous Phyllites and Crystalline Schists.	Serpentine
Method of Design	Tolke and Models	Elastic Arch	Elastic Arch
Purpose	Power	Power	Power
Ownership	Falck	Falck	Falck
Remarks	Double Curvature Arch - Gravity with Cushion.	Multiple Arch	Single Curvature Arch

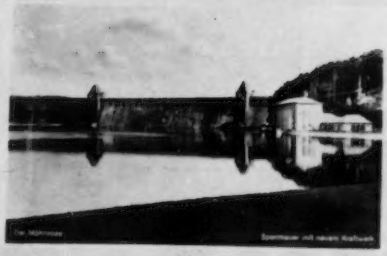
9 VALLA

10 MOLEDANA



13 ZOLA

14 FURAN



15 MÖHNE

RECORDS OF EXPERIENCE

Number	10	11	12
Name	Moledana	Corfino	Turrite Di Gallicano
Location	Italy	Italy	Italy
Height	42.6 M	37.5 M	42.0 M
Top Thickness	2.0 M	1.5 M	3.20 M
Base Thickness	7.61 M	7.0 M	13.50 M
Crest Length	64.3 M	65.0 M	58.5 M
Completion Date	1931	1914	1916
Temperature control during construction	.....	.....	.....
Final Closure	Grouting	No joints	.....
Spillway	Overflow Crest	Siphons	Overflow Crest
Abutment Rock	Crystalline Schists	Serpentine	Grey Limestone
Method of Design	Elastic Arch	Separate Arches	Separate Arches
Purpose	Power	Power	Power
Ownership	Falck	Societa Elettrica Selt-Valdarno	Societa Elettrica Selt-Valdarno
Remarks	Single Curvature Arch	Earthquake Record. See Notes.	Earthquake Record. See Notes.

RECORDS OF EXPERIENCE

Number	13	14	15
Name	Zola	Furan	Mohne
Location	France	France	Germany
Height	140.0 ft.	208.0 ft.	40.3 M
Top Thickness	19.5 ft.	9.0 ft.	6.25 M
Base Thickness	43.0 ft.	174.0 ft.	34.2 M
Crest Length	282.0 ft.	354.0 ft.	650.0 M
Completion Date	1854	1866	1918
Temperature control during construction	.....	.....	.....
Final Closure	.....	.....	.....
Spillway	Channel at Side	Channel at Side	Overflow Weir
Abutment Rock	Hard Limestone	Compact Granite	Shale
Method of Design	Not Known	.....	Gravity Action Alone
Purpose	Water Supply	Water Supply	Water Supply and Power
Ownership	See Notes	Service Des Ponts and Chausees De La Loire	Rurtalsperrenverein
Remarks	Built of Selected Dressed Stone. Reported in perfect condition in 1933.	Constructed of undressed stone with dressed stone faces - bedded in lime mortar. Seepage has decreased from 1.1 to 0.03 gallons per second.	Bombed in 1943 - repaired in 4 months time. Twelve years successful service since. See notes for additional detaile.

upstream face is vertical while the slope of the downstream face is variable. The foundations are a good quality hard limestone and the dam was constructed without cutoff, drains or impervious curtains in the foundations. In consequence there are certain seepage losses under the foundations estimated to be of the order of 10 gallons/sec. when the reservoir is full.

The spillway is a side channel 26' long cut in the right bank abutment. The discharge channel has an extremely steep slope. With a freeboard of only 4 ft. the spillway capacity is insufficient and severe floods sometimes discharge over the crest of the dam.

The intake for the Zola canal is on the right bank 49' below crest level and is controlled by a penstock gate 2' 9" x 3' 0" hand operated with great difficulty by a direct acting winch. Scouring was provided for by two scour valves, one discharging through the right bank abutment 16' 6" below the canal intake, the other through a 19" scour pipe located in a scour gallery situated on the axis of the dam at the base of the wall. Neither of these valves is however serviceable today, as the reservoir has silted up to about 7 feet below the intake level.

Two reports on the dam are available and are quoted below:

1. Note of M. le Sous-Ingeniour des Ponts et Chaussees Coneste on the Zola Dam dated 18th May 1918 (Records of the Department of Ponts et Chaussees, et Aix).

"The Zola Dam continues to behave well. During my last visit I noticed serious streaking over part of the face of the dam due to seepage through cracks and when the dam is full some seepage is visible on the downstream face in the neighbourhood of the scour gallery, which can no longer be used. I have sunk a gauging pit downstream of the structure and this has enabled me to estimate the losses for the whole basin at approximately 15 gallons/sec. but it must be added that the greater part of this takes place through the foundation rock. Not more than 3/4 gal/sec. should pass through the wall itself and there is nothing to indicate that this does not take place at the abutments.

This structure dates from 1853; it does not appear to have suffered any deformation due to temperature effects or to overtopping during severe storms."

2. Report dated May 1933 for the International Commission on Large Dams.

"The dam is still in perfect condition. The losses mentioned in the above note do not seem to have the importance indicated. The dam was emptied during the year 1921 and the upstream face was found to be everywhere in good condition. The intake gate was repaired at that time but should have been replaced by one of a modern type. At the same time a series of scales were placed on the wall to enable water depths to be read.

All in all in spite of its thin section the Zola Dam, doubtless because of its arch form and the solid support given by foundations and abutments, is thoroughly stable and this is confirmed by 80 years of perfect functioning without incident."

Contributed by Mr. J. Martin, Member of the Program Committee—18 November 1955.

14. Furan

"The Furan Dam is a masonry gravity dam constructed in a gorge on

the Furan known as the Gulf of Hell, with which name the dam is often associated. It was commenced in 1863, completed in 1866 and is used as a reservoir for supplying water to the town of St. Etienne in the Department of the Loire. It is maintained and operated by the "Service des Ponts et Chaussees" of the "Department de la Loire."

The dam is constructed of undressed stone extracted from the deviation canal and bedded in lime mortar. The upstream and downstream faces are constructed with variable slopes out of dressed stone. The dam is founded on a hard compact granite. A deviation canal, 5560 ft long, with a capacity of 3510 cusecs commences upstream of the basin of the dam and follows generally the line of top water-level, discharging into the river below the wall. A system of control gates situated in this canal enables floods to be discharged either into the dam or into the river downstream. Additional control measures consist of a discharge tunnel driven in the right bank abutment rock controlled by a valve discharging into the discharge canal, and a right bank side spillway 65 ft long. Intake and scour are provided for by two 15 3/4" pipes and one 8 1/2" pipe drawing off 26.2 ft above the reservoir bottom. The name of the dam, "the Gulf of Hell," has apparently no connection with the possible ultimate destination of its engineers as it has always behaved admirably since its construction. Slight seepage occurs through the downstream face but these losses which were originally 1.1 gals/sec have now dropped to 0.03 gals/sec when the reservoir is full."

Contributed by Mr. J. Martin, Member of the Program Committee—18 November 1955.

15. Mohne—Ownership—Rurtalsperrenverein—Germany
16. Eder—Ownership—Wasserstrombauverwaltung—Germany

The Eder and Mohne dams were bombed in 1943 with the result that a block of about 20 meters in height and 70 m in length with parabolic shape was completely destroyed and swept away in the middle part of the dams. On both sides of the destroyed parts many cracks and fissures appeared.

In the Eder Dam it was evident that there was some arch action before the block was swept away. This could be proved by the displacements in the region of the crest of the dam. Both dams were reconstructed within 4 months by a masonry plug. It was necessary, of course, to remove on both sides masonry masses in very bad state, so that the breadth of the plugs exceeded by 20 meters that of the hole. The many cracks and fissures remaining were artificially closed and grouted. Both dams were in full service since 12 years and proved practically safe and water-tight.

Contributed by Dr. Ing F. Tolke, Direktor Otto-Graf-Institutes—Stuttgart, Germany.—April 4, 1956.

17. Langli Dam—Ownership—Oslo Vann-og Kloakkvesen—Norway.

The dam has been in service only for 15 years. No sign of damage has occurred on the dam during these years. Some small cracks which occurred in the limit between rock and concrete in the beginning and where we got some moisture on the downstream side, have tightened during the years. In the left abutment we very soon got a vertical crack. This crack has never closed. The dam at the crack is about 16 m high. There is air on both sides of this part of the abutment so there is no leakage through the crack. It shows however that this part of the abutment does not act as an abutment at all. Downstream 0.8 m from the arch is placed the insulating wall to protect the dam

against frost damage in winter when the temp. may go down to -25 to 30°C. The insulating wall is 15 cm (6") thick and is made of reinforced concrete which has, of course, very small insulating ability. The important thing is, that we are getting a space between the dam and this wall with still air and it prevents frost damage. Without this wall we certainly would have got frost damage. This is clearly shown on a lot of small intakes and other constructions where the insulating wall is neglected. The rock is tight and no leakage is observed.

The oldest of the arch dams in Norway is the dam at Nedre Skjerkavann which serves as intake dam for the Skjerka Hydro Electric Power Plant. The dam has a height of 16 m over the whole length. The radius is 54.5 m. The dam is made of reinforced concrete and completed 1932. Insulating wall of reinforced concrete 15 cm thick. The dam site is on the level 605 meters. There is no frost damage on the dam but for the plaster on the overflow which has had to be renewed in some places. Experience shows that plaster is not satisfactory on dams or other water constructions. Other arch dams, from the years before and after the Second World War, give the same experience. The dams are poured with a concrete mix of 350-400 kg cement to each m3 concrete. In the insulating walls we only use 300 kg cement. On account of the low temperature during the winter and the acid water in the Norwegian rivers it is not advisable to have lower cement content in our dams without getting bad damage."

Contributed by Ingenior Chr. F. Groner, Oslo, Norway—November 26, 1955.

18. Walters—Ownership—Carolina Power and Light Company.

"Spillway performance has been remarkably satisfactory. There has been little concrete maintenance to date. However, downstream face has been damaged by freezing and thawing action. Also, there are signs of concrete growth; this will receive more study in the near future."

Contributed by Mr. R. W. Gunwaldsen, Hydraulic Engineer, Ebasco Services. March 3, 1955.

19. Angostura—Ownership—Property of the Nation—Mexico.
20. Calles
21. Pabellon

Reports prepared by Mr. Mariano Trejo, Civ. Eng., Assistant of the Engineer in Chief of Irrigation—October 1955.

22. Colimilla—Onwership—New Chapala Electric Company, Mexico.

Report prepared by Eng. Eduardo Rojas G.—October 1955.

These Mexican records were submitted by Mr. Aurelio Benassini, Engeniero en Jefe de Irrigacion y Control de Rios—Secretaria de Recursos Hidraulicos.—Through Mr. Julian Hinds, Member of the Program Committee.

23. Pacoima—Ownership—Los Angeles County Flood Control District.

"Open steel penstocks installed in dam during construction, gates installed on penstocks at completion. Central section concrete pours always lower than mean elevation of finished concrete arch during construction. Reservoir has been full to 11.7 feet above lower spillway crest. Some measurable seepage has occurred at abutments. Abutments have been grouted to some extent on 9 different occasions. No evidence of cracks or deterioration of

RECORDS OF EXPERIENCE

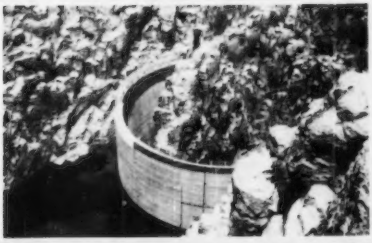
Number	16	17	18
Name	Eder	Langli-Dam	Walters
Location	Germany	Norway	U. S.
Height	48.0 M	37.1 M	207.0 ft.
Top Thickness	5.8 M	0.70 M	16.0 ft.
Base Thickness	36.5 M	1.70 M	40.5 ft.
Crest Length	400.0 M	65.0 M	800.0 ft.
Completion Date	1914	1940	1930
Temperature control during construction	.....	.....	.....
Final Closure	.....	.....	.....
Spillway	Overflow Weir	Overflow	Overflow Crest
Abutment Rock	Shale	Granite	Quartzite
Method of Design	Gravity Action Alone	Trial-Load	Separate Arches
Purpose	Flood Control - Water Supply - Navigation Power	Water Supply	Power
Ownership	Wassestrombauver-Waltung	Oslo Van-Og Kloakkvesen	Carolina Power & Light Company
Remarks	Bombed in 1943 - repaired in 4 months time. Twelve years successful service since. See notes for additional details.	Protected against frost damage by an insulating wall enclosing an air space. No sign of damage in 15 years of service.	Spillway performance remarkably satisfactory. Little concrete maintenance. Some frost damage and signs of concrete growth. See notes.

17 LANGLI

18. WALTERS

19 LA ANGOSTURA

20 CALLES



21 PABELLON

22 COLIMILLA

RECORDS OF EXPERIENCE

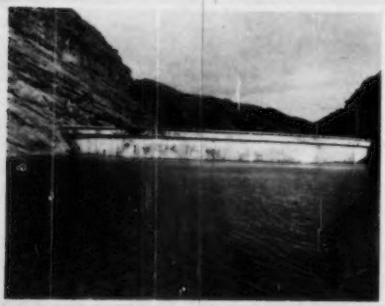
Number	19	20	21
Name	Angostura	Calles	Pabellon
Location	Mexico	Mexico	Mexico
Height	302.0 ft.	220.0 ft.	105.0 ft.
Top Thickness	11.5 ft.	9.8 ft.	3.94 ft.
Base Thickness	101.7 ft.	36.0 ft.	8.20 ft.
Crest Length	584.0 ft.	918.0 ft.	246.0 ft.
Completion Date	1942	1931	1932
Temperature control during construction	Low Heat Cement-Embedded Pipe Cooling	.....	.....
Final Closure	Grouting	.....	.....
Spillway	Overflow	Overflow	Overflow Crest
Abutment Rock	Riolita	Riolita	Riolita
Method of Design	Trial-Loads	Trial-Loads	.....
Purpose	Flood Control, Power, Irrigation.	Power, Irrigation.	See remarks.
Ownership	Property of the Nation.	Property of the Nation.	Property of the Nation.
Remarks	.....	.....	Purpose - Deviation for the Calles Dam.

RECORDS OF EXPERIENCE

Number	22	23	24
Name	Colimilla	Pacoima	Gene Wash
Location	Mexico	U. S.	U. S. Calif.
Height	321.0 ft.	372.0 ft.	138.0 ft.
Top Thickness	11.5 ft.	10.4 ft.	5.0 ft.
Base Thickness	32.8 ft.	99.2 ft.	26.5 ft.
Crest Length	.....	640.0 ft.	430.0 ft.
Completion Date	1949	1929	1938
Temperature control during construction	None	None	Yes
Final Closure	Grouting in upper parts - Closure plug in lower parts.	.....	Grouting
Spillway	Overflow Crest	Tunnel with chute	Off Channel
Abutment Rock	Riolita	Grano-Diorite	Red Sandstone
Method of Design	Separate Arches	Trial-Load	Trial-Load
Purpose	Power	Flood Control, Water Conservation.	Water Supply
Ownership	New Chapala Electric Company	Los Angeles County Flood Control District	Metropolitan Water District of Southern California
Remarks	.....	Some seepage at abutments. Abutments grouted on 9 occasions. No evidence of cracks or deterioration of concrete in structure.	Service experience similar to Copper-Basin but effects of expansion somewhat more moderate. No significant structural impairment.

23 PACOIMA

24 GENE WASH



25 COPPER BASIN

26 ROOSEVELT

27 HORSE MESA

28 MORMAN FLAT

RECORDS OF EXPERIENCE

Number	25	26	27
Name	Copper Basin	Roosevelt	Horse Mesa
Location	U. S. Calif.	U. S. Arizona	U. S. Arizona
Height	210.0 ft.	280.0 ft.	300.0 ft.
Top Thickness	5.0 ft.	16.0 ft.	8.0 ft.
Base Thickness	35.0 ft.	184.0 ft.	45.0 ft.
Crest Length	254.0 ft.	723.0 ft.	660.0 ft.
Completion Date	1938	1910	1927
Temperature control during construction	Yes	None	None
Final Closure	Grouting	.....	.....
Spillway	Concrete Ogee	Side Channel	Two Side Channels and One Tunnel.
Abutment Rock	Red Sandstone	Sandstone	Rhyolite
Method of Design	Trial-Load	Gravity	Trial-Load
Purpose	Water Supply	Power, Irrigation	Power, Irrigation
Ownership	Metropolitan Water District of Southern California	Salt River Project - Arizona	Salt River Project - Arizona
Remarks	Alkali-aggregate expansion Upstream movement of 5.5 inches at top center of dam. Expansion has ceased. Concrete strength good. No significant structural impairment.	This dam has required practically no maintenance.	Service experience good.

concrete in structure. All construction joints are tight."

Report prepared by Mr. Paul Baumann, Assistant Chief Engineer, Los Angeles County Flood Control District.—February 6, 1956.

24. Gene Wash—Ownership: The Metropolitan Water District of Southern California.

25. Copper Basin

"The service experience of Gene Wash dam is generally similar to that of Copper Basin dam, although the effects of expansion in the concrete appear to be somewhat more moderate.

Observations of deflection and elevational changes have been made on Gene Wash dam, but have not been continued as systematically as in the case of Copper Basin dam. For information relating to performance of concrete in Gene Wash dam, reference may be made to the reports listed in the supplementary statement for Copper Basin dam.

As in the case of Copper Basin dam, no significant structural impairment of Gene Wash dam has been detected.

The aggregates used in the concrete in Copper Basin dam were derived from deposits in the bed of the Bill Williams River, in common with the aggregates for Parker and Gene Wash dams. Subsequent developments including comprehensive laboratory investigations, disclosed that these aggregates contained a significant proportion of material reactive to cement, or more particularly to the alkali in cement. The cement used in these dams was purchased under specifications which did not include special limitation of the alkali content, and was moderately high in alkali. The reactivity of the aggregates resulted in the formation of a silica gel wherever there was sufficient residual moisture in the concrete, accompanied by a slow but persistent increase in volume of the interior mass of concrete. The dryer concrete at and just below the surfaces above water was disrupted by the expansion of the interior mass, giving rise to the familiar random-pattern cracking. These cracks, a few of which were up to 1/2 of an inch in width at the surface, rarely extended more than a foot or so into the concrete.

Placement of concrete in Copper Basin dam was completed May 10, 1938, grouting of the vertical joints between the blocks was completed December 3, 1938, and filling of the reservoir with water was started January 29, 1939. Immediately following completion of the dam a point, designated P-2, was set on the parapet near the center of block C, and near the center of the uppermost arch, and a line of sight designated reference Line P, was established between points on the rock near each abutment.

The maximum downstream position of point P-2 was observed March 3, 1939, after the reservoir was full. The radial movement of point P-2 during the reservoir filling was 0.22 of an inch, which is consistent with the estimated deflection of the top arch due to water loading on the dam.

Sporadic observations on point P-2 were continued, and an unexpected upstream movement was recorded, which by October 1942 had attained a magnitude of 2.35 inches from the farthest downstream position under water load as observed March 3, 1939. During 1942 a plumb line was erected on the downstream face of the dam and points were set in the face for measurement of deflections at elevations 1024, 1000, 980, and 960, as well as at the parapet at elevation 1041.5. Access to the plumb line is by a ladder erected on the face of the dam. The plumb line weight is suspended in a receptacle full of oil, to dampen vibration. A comprehensive survey of reference points was

made between October 21 and 26, 1942. Systematic observations have been made at regular intervals beginning October 1942.

The total movement of point P-2 from its maximum downstream position under water load on March 3, 1939, to its maximum upstream position during recent summers, amounts to 5.5 inches. The annual increment was practically constant from 1939 to 1946 at 0.54 of an inch per year. From 1946 to 1952 annual increment decreased, and there has been no cumulative movement whatever since 1952.

Tests of cores cut from Copper Basin and Gene Wash dams during the period between 1940 and 1950 show that no appreciable retrogression in strength or elastic modulus of the concrete has occurred in the two dams. No evidence of significant structural impairment has been discovered."

Contributed by Mr. Robert B. Diemer, General Manager & Chief Engineer, The Metropolitan Water District of Southern California—September 15, 1955.

26. Roosevelt—Salt River Project Agricultural Improvement and Power District.—Arizona
27. Horse Mesa
28. Mormon Flat
29. Stewart Mountain
30. Bartlett

"Roosevelt dam has required practically no maintenance.

There have been no problems experienced with Horse Mesa dam. There is some evidence of alkali aggregate reaction, but there has been no appreciable movement.

There have been no structural problems with Mormon Flat dam.

Alkali aggregate reaction has occurred in the concrete of Stewart Mountain dam. Expansion of the concrete has caused the top of arch in the central portion to move 5.5" upstream. The dam has separated from the powerhouse. An additional mass of concrete has been placed on the downstream face of one abutment to provide reinforcement. Additional grouting of the dam has been done to seal up cracks and construction joints to reduce flow of water past the face of the dam. The object was to reduce the amount of water in the mass and thus the rate of alkali aggregate reaction. Rate of growth has decreased during the last few years.

Reports by Mr. C. H. Whalin—Supervisory Engineer, Civil Division—Salt River Project Agricultural Improvement and Power District.—February 10, 1956.—Transmitted by Mr. T. M. Morong, Chief Engineer to Mr. Julian Hinds, Member of Program Committee.

31. Arrowrock—Ownership—United States Government.
32. Buffalo Bill (Shoshone)
33. Clear Creek
34. Deadwood
35. Gerber
36. Gibson
37. Hoover
38. Owyhee
39. Parker
40. Pathfinder

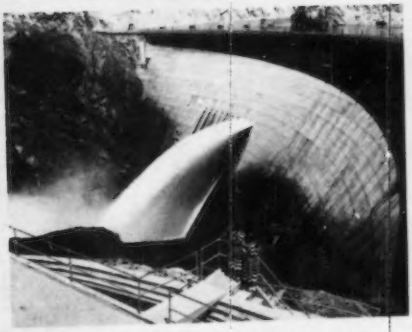
Arrowrock dam was built of sand-cement concrete which had very low frost resistance qualities. Downstream face was badly damaged by frost

RECORDS OF EXPERIENCE

Number	28	29	30
Name	Mormon Flat	Stewart Mountain	Bartlett
Location	U. S. Arizona	U. S. Arizona	U. S. Arizona
Height	224.0 ft.	207.0 ft.	283.0 ft.
Top Thickness	8.0 ft.	8.0 ft.	2.34 ft.
Base Thickness	20.0 ft.	36.0 ft.	7.0 ft.
Crest Length	380.0 ft.	1260.0 ft.	800.0 ft.
Completion Date	1925	1930	1939
Temperature control during construction	None	None	Spray Cooling
Final Closure	.....	Grouting	.....
Spillway	Side Channel and Chute	Side Channel	Chute
Abutment Rock	Rhyolite	Gravity Buttresses	Concrete Buttresses
Method of Design	Separate Arches	Trial-Load	.....
Purpose	Power, Irrigation	Power, Irrigation	Irrigation
Ownership	Salt River Project - Arizona	Salt River Project - Arizona	Salt River Project Arizona
Remarks	Service experience good.	Alkali-aggregate expansion. Upstream movement of 5.5 inches at top center of dam. Repairs by grouting and strengthening of buttress. Rate of growth has decreased in last few years.	Multiple arch. No leakage. No maintenance.

29 STEWART MTN

30 BARTLETT



31. ARROWROCK

32. BUFFALO BILL

33 CLEAR CK.

34 DEADWOOD

RECORDS OF EXPERIENCE

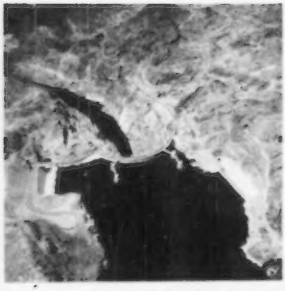
Number	31	32	33
Name	Arrowrock	Buffalo Bill (Shoshone)	Clear Creek
Location	U. S. Idaho	U. S. Wyoming	U. S. Washington
Height	354.0 ft.	325.0 ft.	80.0 ft.
Top Thickness	16.0 ft.	10.0 ft.	3.0 ft.
Base Thickness	223.0 ft.	108.0 ft.	10.0 ft.
Crest Length	1150.0 ft.	200.0 ft.	404.0 ft.
Completion Date	1915	1910	1914
Temperature control during construction	None	None	None
Final Closure	Concreted Shafts	.....	.....
Spillway	Side Channel	Concrete Weir and Tunnel	Overflow Rock Cut Channel
Abutment Rock	Granite	Granite	Igneous Rock and Shale
Method of Design	.....	Trial-Load	Cylinder Formula
Purpose	Water Supply	Water Supply and Power	.....
Ownership	U. S. Government	U. S. Government	U. S. Government
Remarks	Built of sand-cement concrete. Frost damage. New face slab constructed 1937 and dam raised 5 feet. Face slab cracked in 1950.	Concrete in excellent condition in 1948.	Built to height of 50 feet in 1914. Increased 21 feet in 1918. Dam leaks through construction joints. Concrete is sound.

Number	34	35	36
Name	Deadwood	Gerber	Gibson
Location	U. S. Idaho	U. S. Oregon	U. S. Montana
Height	165.0 ft.	88.0 ft.	195.5 ft.
Top Thickness	9.0 ft.	5.0 ft.	15.0 ft.
Base Thickness	62.0 ft.	17.85 ft.	87.0 ft.
Crest Length	749.0 ft.	485.0 ft.	960.0 ft.
Completion Date	1931	1925	1929
Temperature control during construction	.....	None	None
Final Closure	Grouting	Closure Plugs	Grouting
Spillway	Overflow Section	Overflow Weir at Center	Glory Hole
Abutment Rock	Granite	Lava Flows and Conglomerate	Limestone
Method of Design	Trial-Load	.....	Trial-Load
Purpose	Water Supply	Water Supply	Water Supply
Ownership	U. S. Government	U. S. Government	U. S. Government
Remarks	Some grout headers frozen at time of grouting in March 1931. Joints only partially grouted. Some leakage at joints. Frost damage. Spillway lip damaged by icicles.	Frost action on seepage water caused deterioration of concrete. One closure plug shows disintegration of concrete.	Dam in good condition in 1948. Minor seepage through a few joints.

ASCE GLOVER 1217-25

35 GERBER

36 GIBSON



37 HOOVER

38 OWYHEE

39 PARKER

40 PATHFINDER

RECORDS OF EXPERIENCE

Number	37	38	39
Name	Hoover	Owyhee	Parker
Location	U. S. Nevada-Arizona	U. S. Oregon	U. S. Arizona-California
Height	726.4 ft.	417.0 ft.	320.0 ft.
Top Thickness	45.0 ft.	30.0 ft.	39.5 ft.
Base Thickness	660.0 ft.	265.0 ft.	100.0 ft.
Crest Length	1244.0 ft.	833.0 ft.	856.0 ft.
Completion Date	1934	1932	1938
Temperature control during construction	Embedded Pipe Cooling System.	None	Embedded Pipe Cooling System.
Final Closure	Grouting	Grouting	Grouting
Spillway	Side Channel	Glory Hole	Overflow Crest
Abutment Rock	Andesite and Latite Breccia	Felsitic, Rhyolite and Glassy Lavas	Gneiss Granite
Method of Design	Trial-Load and Models	Trial-Load	Trial-Load
Purpose	Flood Control, Water Supply, Power	Water Supply	Flood Control, Water Supply, Power
Ownership	U. S. Government	U. S. Government	U. S. Government.
Remarks	Dam in good condition.	Minor weathering at crest. Evidence of alkali reaction. Mass concrete reported good.	Alkali-aggregate action present. Slab of concrete broken off during foundation grouting operations. Concrete badly cracked in some places.

action from freezing spray from outlet valves. New face was built in 1937 when height of dam was increased 5 feet. The inspection in 1950 disclosed that the new face slab was cracked badly in some places.

Buffalo Bill dam was built during the period 1905-1910, of cyclopean rubble concrete. This dam and Pathfinder dam were designed by trial-load procedures. From a structural standpoint there is nothing in the history of the dam to indicate any weakness or abnormality of behavior of the arch. Rock slides have damaged some of the appurtenant structures but no harm has been done to the dam.

In 1928 the downstream face of the dam was cleaned of scale, and the concrete was reported to be in excellent condition at that time. Concrete in dam is in excellent condition (1948 report).

Clear Creek dam was built to height of 50 feet in 1914. Additional 21 feet added in 1918. No record of joint treatment. Photographs in Report of April 6, 1925, by W. L. Rowe show joints between arch and thrust blocks but no joints in arch section. Dam leaks through cold joints. Leakage has increased between 1925 and 1948, as shown by photographs.

Recent inspections of the dam disclose that the concrete is fairly sound. There are numerous leaks at the horizontal construction joints and the downstream face has weathered badly in places. The seepage is worst at the construction joint at the original crest.

Deadwood dam was constructed during the period from September 1929 to June 1931. At the time the structure was completed there was an unusually light fall of snow and since it was desirable to catch as much of the early runoff as possible, water was stored several months earlier in the season than had been planned originally. This required that the grouting be performed during February and March of 1931, when the weather was unduly severe. Some of the headers to the grouting system were frozen and attempts to thaw them open were only partially successful. The result was that contraction joint grouting was incomplete. Leaks developed through some of the radial contraction joints and also through some of the horizontal construction joints. In 1938 an attempt was made to stop the seepage by regrouting the contraction joints.

The concrete in Deadwood dam has suffered from frost action. The curbs on top of the dam and the concrete in the wet areas have been badly damaged by frost action. The spillway lip has been damaged by the formation of massive icicles, which increase in weight until they spall large areas of concrete. The roof of the valve house has been damaged similarly. Repairs to the curbs on top of the dam and to the spillway lip were made in 1947 and again in 1953.

Gerber dam was built during the period 1923-1925. Because of the difficulty of obtaining adequate natural materials for concrete, it was necessary to manufacture sand. The use of manufactured sand resulted in concrete which was of low strength in certain portions of the dam.

After the dam was completed and placed in service, seepage developed through horizontal construction joints. This seepage, coupled with freezing weather during winter months, caused considerable disintegration of the concrete. In the fall of 1951, the water surface in the reservoir was lowered and the upstream face of the dam was covered with a waterproof membrane, which eliminated most of the seepage.

The dam was built with closure plugs between the arch and gravity sections. The concrete in the plug at the left end of the arch shows considerable deterioration in places.

Gibson dam was constructed during the period 1926-1927.

The condition of the dam is good. There are only a few minor seeps. There are no structural deficiencies.

Hoover dam was constructed during the period 1930-1935.

Observations for uplift pressure on the base of the dam were made periodically as the reservoir filled. These observations indicated that the uplift pressure beneath the Nevada side of the dam was increasing and that it exceeded that used as a basis for design. Corrective measures were started in 1939 by grouting a deeper and more extensive cut-off curtain slightly downstream from the axis of the dam. This work was followed by the establishing of a new system of foundation drains downstream from the additional grout curtain. Since the program of additional grouting and drainage was performed, there have been no further developments at Hoover dam affecting the behavior of the arch structure."

Owyhee dam was built during the period 1928-1932.

Shortly after the reservoir was filled, considerable leakage developed. The total flow was about 15 cubic feet per second, with most of this being through the left abutment and emerging downstream quite close to the dam. A program of supplemental grouting was started in April 1936 and completed in October 1937. The present flow from seepage is between 2 and 3 cubic feet per second.

Since the completion of the foundation grouting, there has been no further work at Owyhee dam. From a structural standpoint, the arch is in good condition although there is considerable cracking of the concrete. In the upper elevations recent inspections have disclosed that there is considerable evidence of alkali-aggregate reaction, which, combined with the effects of freezing and thawing, has caused disintegration of the curbs, walkways, parapets, and lamp post bases.

Parker dam was built during the years 1934-1938.

While grouting operations were in progress during the construction of the dam, a large slab of concrete, up to 18 inches thick, cracked loose from the face of the dam at the right abutment. This slab was removed and 183 cubic yards of new concrete was placed in the defective area. Dowels were placed in the older concrete to anchor the new concrete. Since that time numerous cracks have developed at various places in the structure, which have been attributed to the reaction of the high alkali cement with the aggregates.

The severely cracked condition of the exposed surface of the dam has been discussed quite extensively in technical writings on concrete design and structural behavior. Surface cracking became noticeable and of serious concern within a relatively short time after completion of the dam. An extensive investigation of the crack development was made in 1940 and 1941 and from this it was concluded that the cracking, although extensive, did not extend very deep. Cores drilled from the concrete when stored in fog room exuded gels, and when tested for compressive strength and modulus of elasticity gave somewhat lower values than expected. Samples of the concrete subjected to cycles of wetting and drying showed great expansive properties.

The alkali-aggregate reaction has caused sufficient expansion of the concrete to make operating the gates on top of the dam difficult, and to require considerable readjustment in order to obtain sufficient clearance. This work was completed in 1942. An inspection made in 1954 revealed no further difficulties with the arch structure except for some minor leaks.

Pathfinder dam is a masonry arch dam built during the period 1905-1909.

41 CHEESEMAN

42 ELEVEN MI. CANYON

43 MOVIE

45 MERWIN

46 BAKER R.

47. DIABLO

RECORDS OF EXPERIENCE

Number	40	41	42
Name	Pathfinder	Cheesman	Eleven Mile Canyon
Location	U. S. Wyoming	U. S. Colorado	U. S. Colorado
Height	214.0 ft.	232.0 ft.	142.0 ft.
Top Thickness	10.9 ft.	18.0 ft.	15.0 ft.
Base Thickness	96.5 ft.	167.0 ft.	53.1 ft.
Crest Length	432.0 ft.	634.0 ft.	496.0 ft.
Completion Date	1909	1904	1932
Temperature control during construction	None	None	None
Final Closure	.....	None	Grouting
Spillway	Weir at North End of Dam	Around End	Around End
Abutment Rock	Granite	Granite	Biotite Granite
Method of Design	Trial-Load	Gravity	Trial-Load
Purpose	Water Supply	Water Supply	Water Supply
Ownership	U. S. Government	City and County of Denver	City and County of Denver
Remarks	Dam in good condition. Faces of Ashlar Masonry. Cyclopean Masonry in the Interior.	No cracks-no leaks-no repairs. Masonry construction. Loads carried by Arch Action were estimated.	No cracks-no leaks-no repairs.

Number	43	44	45
Name	Moyie	Lake Lanier	Merwin (Ariel)
Location	U. S. Idaho	U. S. North Carolina	U. S. Oregon
Height	53.0 ft.	62.0 ft.	313.0 ft.
Top Thickness	2.0 ft.	2.0 ft.	19.4 ft.
Base Thickness	5.33 ft.	12.0 ft.	93.0 ft.
Crest Length	154.0 ft.	236.0 ft.	728.0 ft.
Completion Date	1924	1925	1931
Temperature control during construction	.....	.....	Cooling water in vertical holes.
Final Closure	.....	.....	Closure Plugs
Spillway	Timber lined cut to side stream.	Overflow Crest	Separate Overflow Section.
Abutment Rock	Stratified Quartzite dipping 30° to 45° downstream.	Folded Granitic Rock	Andesite and Basalt
Method of Design	.....	.....	Trial-Load
Purpose	Power and Milling	Impoundment for a pleasure lake.	Power
Ownership	Abandoned	.....	Pacific Power & Light Company
Remarks	Unprecedented flood destroyed spillway and destroyed east abutment. Dam still standing in 1954. Damage was never repaired. Project in state of abandonment.	Undetected foundation weakness caused loss of west abutment. Arch did not fail. Repaired by addition of gravity abutment, spillway section and addition of earth fills converting it to core type dam. It has successfully impounded water since its repair.	Dam remarkably tight at both vertical and horizontal construction joints.

RECORDS OF EXPERIENCE

Number	46	47	48
Name	Baker River	Diablo	Ross
Location	U. S. Washington	U. S. Washington	U. S. Washington
Height	285.0 ft.	389.0 ft.	540.0 ft.
Top Thickness	12.0 ft.	16.0 ft.	33.0 ft.
Base Thickness	120.0 ft.	146.0 ft.	208.0 ft.
Crest Length	450.0 ft.	1180.0 ft.	1300.0 ft.
Completion Date	1925	1930	1949
Temperature Control during construction	None	None	See notes.
Final Closure	Closure Plug	Grouting	Grouting
Spillway	Overflow Crest	Overflow Abutment Gravity Section	Overflow Crest with Deflector Hoods
Abutment Rock	Limestone	Granite Gneiss	Granite Gneiss
Method of Design	Trial-Load	Separate Arches	Trial-Load
Purpose	Power	Power	Flood Control, Power
Ownership	Puget Sound Power & Light Company	City of Seattle	City of Seattle
Remarks	Overflow spillway passed 21,712 cfs. in 1949. Spillway designed for 36,000 cfs.	In continuous service since completion except during construction joint grouting in 1950-1951.	In continuous service since completion. (This dam built in steps - See notes.)

The dam was constructed of cyclopean, coarse grained granite stones with a maximum size of 10 tons. These stones were bedded in cement mortar and the vertical joints filled with granite spalls. The ashlar facing stones were laid in horizontal courses with 2-inch maximum joints. As built, the dam contains 48 percent rock, 39 percent concrete, and 13 percent mortar. The stone for the masonry was quarried from the granite of the canyon walls and concrete aggregate was obtained from the river bed. The upper 27 feet of the dam was reinforced horizontally with steel. There is nothing in the history of Pathfinder dam which indicates any abnormality or deficiency in structural behavior. Repeated inspections of the structure have disclosed that the masonry is in excellent condition, with no leakage showing on the downstream face.

During the winter seasons, the operating personnel have always been bothered by accumulations of ice on the canyon walls downstream from the dam due to freezing of seepage water passing through the abutments. In 1949 a program of grouting the abutments was undertaken with the result that while the seepage was reduced it was not entirely eliminated. The grouting was successful for the reservoir elevation at the time the grouting was done, but the reservoir elevation increased in 1950 and new leaks appeared at the higher elevations in both abutments. No additional grouting has been done since 1949."

Data on Bureau of Reclamation dams supplied by Mr. L. N. McClellan, Assistant Commissioner and Chief Engineer.

Extract from a letter of April 2, 1956 from Mr. Wayne A. Perkins—Consulting Engineer.

Clear Creek dam was designed as a variable radius type, vertical at crown at the intrados, and vertical at the theoretical abutment at the extrados, this point being the contact between the arch and the gravity abutments. This criterion of design was set up to avoid overhang at any point. Stresses were determined by the cylinder formula. Limiting stresses set by the consulting board were 25 tons compression and 50 lbs tension.

After we moved in to start construction we discovered that the prepared design could not be used as the abutments were not adequate, necessitating a new design at a site farther upstream, which was made by the field crew on the ground and under pressure in order not to hold up the construction crew. When this dam was built the science of arch dam analysis was not well developed, and most of the earlier arches were designed by the cylinder formula which was used by Mr. D. C. Henny who supervised the first design and was a consultant throughout the construction.

41. Cheesman—Ownership—City and County of Denver.
42. Eleven Mile Canyon

Lake Cheesman dam was completed in 1904.

There have been no cracks, no leaks, no repairs except to outlet valves. There has always been a small seep through rock formation in canyon wall near end of dam.

Eleven Mile Canyon dam was completed on October 30, 1932. The contraction joints were grouted later. There have been no cracks, no leaks, no repairs, but should have new roadway paving and repair on face of highline due to frost action. There has always been a small seep through rock formation in canyon wall near end of dam.

Contributed by Mr. D. D. Gross, Consulting Engineer.

43. Moyie—Present ownership not determined.

Construction practically complete in 1924.

The abutment rock was stratified quartzite dipping sharply (30°-45°) downstream. Left abutment was in a sharp narrow ridge between the Moyie River and a side stream. The spillway was a timber lined cut 24 feet wide, 12 feet deep and 50 feet long discharging into the side stream on the left abutment.

According to a report by the U.S. Forest Service information reached their office about May 1, 1925 that three small dams had failed on the Moyie River in Canada due to high water. The resulting flood took out the highway bridge at Meadow Creek above the Cynide Gold Mining Company Project. The debris and flood water apparently tore out the timber lining of the spillway. Water cut out the entire spillway section and entirely destroyed the abutment at the east, left, end of the concrete arch dam. According to the report the arch section of the dam was apparently undamaged except for a crack near the east abutment.

Report prepared by Mr. John F. Mangan, Civil Engineer, Spokane, Washington.

44. Lake Lanier

Lake Lanier dam was completed in March 1925.

Extract from a letter of January 21, 1955, from Mr. Shannon Meriwether, Tryon, North Carolina.

"The dam was never a power dam; it simply impounded the water for a pleasure lake. It has successfully impounded the water since its repair.

Local inquiry has given me this information:

About two thirds of the original dam served as a spillway—there was no separate spillway. Failure was due to undermining at one end. It is said that this foundation failure resulted from the fact that this section rested on a large boulder—say forty feet across, instead of on hard bed-rock. Reports are that there were only a few borings. But even if borings had been carefully and thoroughly made, the boulder and its surrounding clay or detritus might not have shown up as such. In this locality the rock is very ancient, all granitic, and never stratified. Its folding, fracturing, and metamorphosis is almost beyond belief. At any rate, pressure from the impounded water loosened the material around the boulder and it moved, letting the section of the dam go out. I cannot determine how it moved, whether it sunk and overturned or just what did happen. Repair was made by dewatering that end of the dam and removing everything to a more certain bed-rock, and then building a new heavy gravity section, designed as a spillway, with a bridge over it. Then, as an added precaution, earth fill was added on both sides of that portion of the dam which had not failed, until the dam became in effect a core type dam."

Reference:

The October 14, 1926 issue of Engineering News-Record contains an account of the abutment failures at both the Moyie and Lake Lanier sites.

45. Merwin (Ariel)—Pacific Power and Light Company.

Merwin dam was completed on April 18, 1931.

"The dam was and is remarkably tight at both horizontal and vertical construction joints."

Report prepared by Mr. E. Robert de Luccia, Vice President and Chief Engineer, Portland, Oregon—March 7, 1955.

48 ROSS

49 KARAPIRO

50 MARAETAI

52 FRASER

53 MANORBURN

54 UP^st CORDEAUX #1

RECORDS OF EXPERIENCE

Number	49	50	51
Name	Karipiro	Maraetai	Poolburn
Location	New Zealand	New Zealand	New Zealand
Height	170.0 ft.	284.0 ft.	100.0 ft.
Top Thickness	8.0 ft.	10.0 ft.	4.0 ft.
Base Thickness	50.0 ft.	50.0 ft.	30.0 ft.
Crest Length	576.0 ft.	436.0 ft.	400.0 ft.
Completion Date	1947	1952	1931
Temperature control during construction	Open slots	Embedded cooling pipes	4 ft. wide keyed slots
Final Closure	Slots filled	Grouting	Slots filled
Spillway	Separate Overflow Crest	Tunnel	Overflow Crest
Abutment Rock	Greywacke	Ignimbrite	Mica-Schist
Method of Design	Trial-Load	Separate Arches and Trial-Load	Cylindrical Arch
Purpose	Power	Power	Irrigation
Ownership	New Zealand Government	New Zealand Government	New Zealand Government
Remarks	Service experience - satisfactory.	Service experience - satisfactory.	.....

RECORDS OF EXPERIENCE

Number	52	53	54
Name	Fraser	Manorburn	Upper Cordeaux No. 1
Location	New Zealand	New Zealand	Australia, N.S.W.
Height	107.0 ft.	90.0 ft.	46.5 ft.
Top Thickness	2.5 ft.	3.0 ft.	3.5 ft.
Base Thickness	20.0 ft.	25.0 ft.	11.6 ft.
Crest Length	450.0 ft.	380.0 ft.	540.0 ft.
Completion Date	1937	1915	1902
Temperature control during construction	.....	None	.....
Final Closure	.....	.....	.....
Spillway	Overflow Crest	Overflow	Overflow
Abutment Rock	Slabby-Schist	Schist	Basalt
Method of Design	Cylindrical Arch	Cylindrical Arch	Thin Cylinder
Purpose	Irrigation	Irrigation	Water Supply
Ownership	New Zealand Government	New Zealand Government	Metropolitan Water, Sewerage & Drainage Board - Sydney.
Remarks	Service experience - satisfactory.	No details available.	No major repairs in 54 years of service.

46. Baker River—Puget Sound Power and Light Company.

Baker River dam was completed in 1925.

"The spillway of this arch type dam was designed with a discharge capacity of 36,000 c.f.s. It has been found to be adequate for any flood which has occurred during the past 25 years. The worst flood on record for this dam occurred on November 27, 1949 when 21,712 C.F.S. was recorded."

This report was prepared by Mr. A. L. Pollard, Operating Manager—Seattle, Washington.

47. Diablo—City of Seattle
48. Ross

Diablo dam was completed November 1930.

This dam has been in continuous service since completion except during construction joint grouting. Construction joint and foundation grouting and spillway paving to stop rock erosion was part of rehabilitation and improvement program in 1950 and 1951. Original design was based on the assumption that the arch rings carried the entire water load to the abutments with no transfer of load to foundation by cantilever or vertical beam action. A trial load analysis was made in 1949.

Ross dam was completed in June, 1949.

This dam was designed for construction in four steps. The First Step from foundation rock at elevation 1075 to elevation 1365. The Second Step to elevation 1550. The Third Step to elevation 1615. The Ultimate Height to elevation 1733. In construction the second and third steps were carried through in one continuous operation.

Report submitted by Mr. C. E. Shevling, Skagit Project Engineer, Seattle, Washington.

49. Karapiro—New Zealand Government.
50. Maraetai
51. Poolburn
52. Frazer
53. Manorburn

Karapiro dam was completed in April, 1947.
Service experience: Satisfactory—nothing unusual.
Maraetai dam was completed in December, 1952.
Service experience: Satisfactory, nothing to report.
Poolburn dam was completed in 1931.
Service experience: No details available.
Frazer dam was completed in 1937.
Service experience: Satisfactory.
Manorburn dam was completed in 1915.
Service experience: No details available.
Reports prepared by Mr. F. R. Askin, Chief Designing Engineer (Hydro). Hydro-electric Design Office, Ministry of Works, Wellington, New Zealand. 31.12.55.

54. Upper Cordeaux No. 1. Metropolitan Water, Sewerage & Drainage Board, Sydney, N.S.W.
55. Upper Cordeaux No. 2.
56. Warragamba
57. Bargo.

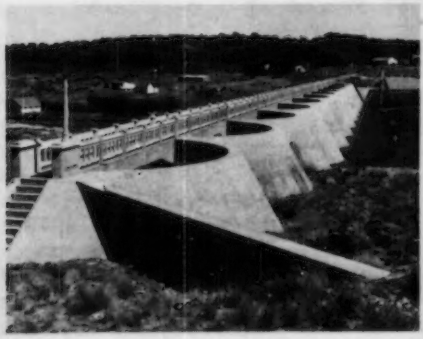
55 UPPER CORDEAUX #2

56 WARRACAMBA

57 BARGO

58 BAROSSA

59. MT BOLD



71 MIENA

Number	55	56	57
Name	Upper Cordeaux No. 2	Warragamba Balance Reservoir	Bargo
Location	Australia, N.S.W.	Australia, N.S.W.	Australia, N.S.W.
Height	70.0 ft.	54.0 ft.	43.0 ft.
Top Thickness	3.5 ft.	3.0 ft.	4.0 ft.
Base Thickness	38.0 ft.	14.25 ft.	13.6 ft.
Crest Length	815.0 ft.	358.8 ft.	137.0 ft.
Completion Date	1915	1940	1897
Temperature control during construction	.....	None	.....
Final Closure	.....	Closure Plug	.....
Spillway	Overflow Spillway	Separate Side Spillway	Overflow
Abutment Rock	Sandstone	Sandstone	Sandstone
Method of Design	Thin Cylinder	Thin Cylinder	Thin Cylinder
Purpose	Water Supply	Water Supply	Water Supply
Ownership	Metropolitan Water, Sewerage & Drainage Board - Sydney.	Metropolitan Water, Sewerage & Drainage Board - Sydney.	Metropolitan Water, Sewerage & Drainage Board - Sydney.
Remarks	Structure sound. No major repairs have been necessary.	Slight seepages at joints. No remedial measures have been required.	In continuous use for 59 years. Some cracking and seepage in evidence but no major repairs are considered necessary.

RECORDS OF EXPERIENCE

Number	58	59	60	61
Name	Barossa	Mount Bold	Boreenore Creek	Puddle Dock Creek
Location	Australia South Australia	Australia South Australia	Australia, N.S.W.	Australia, N.S.W.
Height	112.75 ft.	177.0 ft.	53.5 ft.	61.0 ft.
Top Thickness	4.5 ft.	12.5, 22.0 & 25.0 ft.	3.5 ft.	3.5 ft.
Base Thickness	41.0 ft.	102.0 ft.	25.5 ft.	24.8 ft.
Crest Length	472.5 ft.	717.5 ft.	405.0 ft.	269.6 ft.
Completion Date	1903	1938	1928	1928
Temperature control during construction	No	None	.....	.....
Final Closure	No Contraction Joints	Grouting	.....	.....
Spillway	None	Dam Overshot	.....	.....
Abutment Rock	See Notes	See Notes	.....	.....
Method of Design	Separate Arches	Separate Arches	.....	.....
Purpose	Water Supply	Water Supply	.....	.....
Ownership	South Australia Government	South Australia Government	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.
Remarks	Dam has given satisfactory service.	Dam has given satisfactory service.	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.

RECORDS OF EXPERIENCE

Number	62	63	64	65
Name	Nattai Creek	Coeypoly Creek	Beardy River	Connors Creek
Location	Australia, N.S.W.	Australia, N.S.W.	Australia, N.S.W.	Australia, N.S.W.
Height	25.1 ft.	62.6 ft.	28.0 ft.	36.75 ft.
Top Thickness	3.5 ft.	3.5 ft.	4.0 ft.	3.5 ft.
Base Thickness	11.1 ft.	23.2 ft.	9.1 ft.	12.3 ft.
Crest Length	421.5 ft.	327.3 ft.	140.0 ft.	365.0 ft.
Completion Date	1931	1932	1932	1934
Temperature control during construction	.....	.....	.....	.....
Final Closure	.....	.....	.....	.....
Spillway	.....	.....	.....	.....
Abutment Rock	Sandstone	Sedimentary	Granite	Diorite
Method of Design	.....	.....	.....	.....
Purpose	.....	.....	.....	.....
Ownership	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.
Remarks	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.

RECORDS OF EXPERIENCE

Number	66	67	68	69
Name	Flat Rock Creek	Mooney Creek	Back Creek	Greaves Creek
Location	Australia, N.S.W.	Australia, N.S.W.	Australia, N.S.W.	Australia, N.S.W.
Height	49.43 ft.	40.5 ft.	44.8 ft.	57.0 ft.
Top Thickness	3.5 ft.	3.5 ft.	3.5 ft.	3.5 ft.
Base Thickness	11.9 ft.	11.0 ft.	15.7 ft.	16.1 ft.
Crest Length	270.0 ft.	158.5 ft.	365.0 ft.	220.0 ft.
Completion Date	1935	1937	1937	1942
Temperature control during construction	.....	.....	.....	.....
Final Closure	.....	.....	.....	.....
Spillway	.....	.....	.....	.....
Abutment Rock	Sandstone	Shale and Sandstone	Shale	Sandstone
Method of Design	.....	.....	.....	.....
Purpose	.....	.....	.....	.....
Ownership	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.
Remarks	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.

Upper Cordeaux dam No. 1 was completed in 1902.

Apart from some leaching of mortar at the overflow section and calcium carbonate deposits at the non-overflow section, this dam is in very good condition after 54 years of service. No major repairs have been carried out. The reservoir is normally kept full.

Upper Cordeaux dam No. 2 was completed in 1915.

The reservoir is normally kept full. There is some leaching out of mortar on upstream face and at overflow section on downstream face; also some calcium carbonate deposits, but structure is quite sound and no major repairs have been found necessary.

Warragamba dam was completed in 1940.

This dam has been in service, mainly as a balance reservoir, on a 48-in. rising main for 16 years. Grouting of joints was provided for but was found to be unnecessary. There are some slight seepages through vertical and horizontal construction joints but no remedial measures have been required.

Bargo dam—The original completion date was 1897; raised 8-ft. 1910; raised further 7-ft. 1947. This dam has been in continuous use for 59 years. Some cracking and seepage are in evidence, but no major repairs are considered necessary.

Reports submitted by Metropolitan Water, Sewerage & Drainage Board, Sydney, N.S.W., Australia. March 1956.

58. Barossa—South Australia Government
59. Mount Bold

Barossa dam was completed in February, 1903.

"The Barossa dam is of particular interest as it was one of the earliest of the thin arch structures in Australia. The abutment rock is argillaceous and arenaceous laminated rock with micaceous shale joints. It was constructed without contraction joints. The dam has given satisfactory service."

Mount Bold dam was completed in 1938.

"The abutment rock is Hard Phyllite with Limestone bands. The dam is of the constant radius arch type with gravity abutments. Puddled clay cut off walls extended from the concrete abutments into the river side slopes for a distance of 250 ft. on the south-easterly side and 150 ft. on the north-westerly side. Foundations grouted. The dam has given satisfactory service."

Report prepared by Mr. J. R. Dridan, C.M.G., B.E., A.M.I.E. Aust., Engineer-in-Chief, Engineering and Water Supply Dept., S.A. Govt., South Australia. May 28, 1956.

60. Boreenore Creek—Department of Public Works, N.S.W. Sydney, Australia.
61. Puddle Dock Creek
62. Nattai Creek
63. Coeypolly Creek
64. Beardy River
65. Connors Creek
66. Flat Rock Creek
67. Mooney Creek
68. Back Creek
69. Greaves Creek
70. Cudgegong River

"All these dams have given satisfactory service and are now in good condition and look like remaining in service for an indefinite future period.

72 CLARK 73 RIDGEWAY

RECORDS OF EXPERIENCE

Number	70	71	72	73
Name	Cudgegong River	Miena	Clark	Ridgeway
Location	Australia, N.S.W.	Tasmania	Tasmania	Tasmania
Height	56.6 ft.	40.0 ft.	199.0 ft.	195.0 ft.
Top Thickness	3.8 ft.	1.0 ft.	15.0 ft.	6.0 ft.
Base Thickness	25.0 ft.	1.9 ft.	80.0 ft.	53.75 ft.
Crest Length	465.0 ft.	1080.0 ft.	1110.0 ft.	232.75 ft. Arch 293 & 203 ft. Abutment
Completion Date	1953	1922	1949	1919
Temperature control during construction	.....	Nil	Cooling with River Water.	.....
Final Closure	.....	.....	Grouting	Closure Plug
Spillway	.....	Nil	Ski-Jump	Weir and Tunnel
Abutment Rock	Granite	Dolerite	Dolerite	Dolerite
Method of Design	.....	Separate Arches	Trial-Load	Not Known
Purpose	.....	Power	Power	Water Supply
Ownership	Department of Public Works, N.S.W.	Hydroelectric Commission of Tasmania.	Hydroelectric Commission of Tasmania.	Corporation of City of Hobart.
Remarks	Dam has given satisfactory service and is now in good condition.	Multiple arch dam. 27 arches, 40 foot span. Few slight leaks sealed by mortar on upstream face during low water. Concrete still very sound.	Minor seepage through a few horizontal construction joints and grout pipes.	Minor leakage on arch face repaired by plugging visual faults on pressure fact.

Some scour has occurred at the downstream toe of some of these dams and, in one case, a plug of clayey material was washed from underneath the foundation of an arch dam, thus emptying the storage. However, all these minor defects have been repaired effectively at little cost."

Reports submitted by Mr. J. M. Main, Director of Public Works—May 23, 1956.

71. Miena—The Hydro-Electric Commission, Tasmania.
72. Clark

Miena dam was completed in 1922.

"Few slight leaks at horizontal construction joints, sealed by mortar on upstream face during low water. Concrete still very sound."

Clark dam was completed in 1949.

"Minor seepage through a few horizontal construction joints and grout pipes. Leakage into gallery through foundation drains less than 10 Imp. gallons per minute with reservoir full."

Report prepared by Mr. G. T. Colebatch, Chief Civil Engineer, The Hydro-Electric Commission, Hobart, Tasmania—5th April, 1956.

73. Ridgeway—Corporation of the City of Hobart, Tasmania.

Ridgeway dam was completed in 1919.

"Minor leakage since construction has persisted at a number of points on the arch face. Reduced by plugging visual faults on pressure face."

Report prepared by Mr. Kenneth V. Harris, Designing Engineer, Hobart City Council, Tasmania.

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POWER DIVISION
Proceedings of the American Society of Civil Engineers

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Note: Paper 1225 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, PO 2, April, 1957.

*There will be no closing discussion to this paper.

Discussion of
"ARCH DAMS: THEIR PHILOSOPHY"

by Andre Coyne
(Proc. Paper 959)

ANDRE COYNE,[1] Hon. M. ASCE.—The remarks of G. S. Sarkaria and George E. Goodall complement most auspiciously the writer's paper on the Philosophy of Arch Dams. Their closeness together gives them yet further interest, for they each cast light upon the subject in very different and even opposite ways.

The writer agrees most willingly with the first discusser, G. S. Sarkaria, that it is most desirable to tidy up the classification of arch dams.

Classification is useful—not to say necessary—both for those who teach and for those who learn, for it helps to get new data into the mind and to keep them in the memory. It forms a framework for exhibiting or for retaining the essential landmarks. From this point of view the pursuit of the simplest and most comprehensive classification therefore presents unquestionable utility. The same is true of the research, analysis and codification of the rules followed and hallowed by great builders up to the present time.

It is here the part it plays stops. A part which, it may be said consists merely in organizing the conquered territory.

Now, to judge by the number and diversity of the examples purposely proposed by the writer, there are still territories awaiting conquest, which will not fail one day to tempt some innovating mind.

In the technical sphere conquest, the realisation of progress, means being able or having the courage to do that which has not as yet been done. It means emerging from the rigid conventional framework or at least extending it, even bursting forth from it, in a word, ceasing to consider it an imprescriptible law of the Medes and Persians. This is of course the idea of the discusser, when he concedes that the present empirical limit of the canyon-shape factor for arch dams can be changed in the future.

Far from "not being considered representative" today's exception, Kariba for instance, can become tomorrow's rule.

This is the reason why the writer himself has always shared the opinion that as things are at present, formal and imperative regulations do more harm than good in the sphere of arch dams, particularly where the conditions of adaptation to the site, methods of calculation and permissible stresses—specially tensile stresses—are concerned.

This all the more if, as the discusser suggests, such criteria result "from a compromise between the various points of view."

On the contrary, the greater the part given to diversity of individual inspiration the better the result, taking into account the variety of sites and working potentialities which the earth offers unceasingly to designers.

Doing without the impassable barriers built up by the regulations boils down to relying principally upon the gifts for observation, the know-how and

1. Inspector General, Cons. Engr., A. Coyne and J. Bellier, Paris, France.

and the good judgment of designers; a thing which is only apparently dangerous for no ruling on construction has ever prevented people from committing accidentally fatal fundamental mistakes.

It can even be maintained without being paradoxical that the illusion of being covered by regulations and exaggerated attachment to tradition can hamper designers in establishing correct diagnoses by inhibiting their capacity and even their desire for apprehending reality. This is the major risk. Precedent has been called humourously the momentum of an early start in a wrong direction.

G. E. Goodall on his side, recalls in a timely way the vigour and fecundity of the great current of pragmatism started in the United States of America by the first arch dam builders under the empire of "holy necessity," that is to say lack of money.

The writer is particularly grateful to him for bringing grist to his mill by quoting fresh examples which the discusser declares to be "obviously horrible," of arch dams which have stood up obstinately to many kinds of causes of destruction, of which some could incidentally, have been obviated.

These examples again, have to be interpreted with care, in the light of modern knowledge of the mechanical behaviour of such structures and the quality of their materials. Here again, the "sixth sense" of diagnosis, the wisdom and good judgment of the designer are the decisive and irreplaceable factors. This is one of the grandeurs of his profession.

The writer associates himself to the full with the very sensible remarks made by the discusser and by Dr. Frederik Vogt on the proper use of calculations, and on the differences revealed by experience between actual and computed stresses. It is for this reason that he persists in believing that it is from the experimental method rather than from analysis that fresh progress can be expected in a sphere which is in full evolution.

The writer was not aware that traces of arch dams built in Africa before the Christian Era had been found. Should such relics exist, they ought not to be attributed to the Arabs, who only started invading Africa in the VIIth Century A.D.

Henri Goblot, a French Engineer residing in Iran, has just informed the writer of his discovery near the town of Koum, of an old masonry dam, 26 m high, curved or rather polygonal in plan. It is well-preserved but now empty. According to local information it was probably built in the middle of the thirteenth century.

Although its shape is not perfect this dam is certainly one of the ancestors of arch dams, the oldest we know up to date.

Discussion of
"HYDRO-ELECTRIC POWER IN THE SOUTHEAST"

by C. P. Lindner and L. F. Johnson
(Proc. Paper 1086)

ROBERT C. PRICE,[1] M. ASCE, and T. G. SWENNES[2].—The paper presented by Mr. Lindner gives an excellent review of the progress made in electric power development in the Southeast. He has very ably outlined the existing power projects, those now under construction and the Federal projects with power installations which are scheduled to be built by 1962.

His review of future projects with hydroelectric power which are planned by the Corps of Engineers brings up several interesting points for discussion, among which is the relationship between this potential power supply and the estimated loads which it would serve. Obviously, engineers engaged in the planning of a power supply must deal with future potential loads and their characteristics in order to complete the power development picture. The writers shall discuss this subject briefly as a continuation, or sequel, to the paper by Mr. Lindner and Mr. Johnson.

Paralleling the phenomenal growth in the development of the power supply in the Southeast during the last decade is, of course, the extraordinary expansion of the demand for electric power. This demand has originated largely from marked industrial reorganization and expansion connected with low-cost power available to industry. Spectacular increases in the power demand have also resulted from a large expansion in domestic uses owing to changes in social habits as the retail consumer became adjusted to increased use of power in the home. Furthermore, the national defense effort since 1950 has created an additional demand of enormous proportions. This demand coupled with unprecedented expansion is currently requiring large-scale increases in the power supply of the Southeast.

These past power requirements are, of course, well known but what about the amount of power which will be required for loads in the future, and what will be its characteristics? Will the demand continue to grow and expand, and will the characteristics of the future loads be suitable for the utilization of the power from all of the planned Federal projects mentioned by Mr. Lindner?

The staff of the Federal Power Commission, in connection with its other functions, makes continuing studies and prepares estimates to answer these and other related questions concerning the developing and marketing of electric power. The results of these studies are not only for Commission use but also for the use of other agencies, including the Corps of Engineers, who are concerned directly with the various phases of planning, design, construction, and operation of Federal hydroelectric power projects. These

1. Regional Engr., Atlanta Regional Office, Federal Power Commission, Atlanta, Ga.
2. Electrical Engr., Atlanta Regional Office, Federal Power Commission, Atlanta, Ga.

studies and estimates are finally reduced to reports on power markets and power values for the purpose of determining whether the new supply can be utilized if constructed, and whether the proposed projects are economically feasible.

Power market surveys with respect to the utilization of any planned power supply involve the forecasting of future loads. These forecasts are divided, generally, into three types, each of which serves a distinctly separate purpose. These types are: (1) "Current" forecasts, which are made by days for short periods of a month or two; (2) "Short-Term" forecasts, made by months for a period of one to four years; and (3) "Long-Range" forecasts, which estimate the loads for a period of 5 to 20 years or longer.

This discussion deals principally with "Long-Range" forecasts, which are intended to show the normal trend of the expected power requirements on the basis of the natural resources, assuming a reasonable rate of development. These long-range forecasts are indispensable in planning the development of the power resources of river basins because they indicate the amount of generating capacity that can be absorbed in a given period and provide a basis for scheduling the individual projects in an orderly and economic manner.

The methods employed in making forecasts for long-range power requirements are necessarily more comprehensive than those which are used in making current or short-range estimates. The electric load increase is a function of the resources and economic potentialities of a region, and these factors must be carefully evaluated in determining the load expansion on a long-range basis.

The total future requirements for the use of electric power in an area is estimated by first evaluating its component parts, such as farm use, residential use, commercial use, and industrial use. From the composite of all these estimated future uses, together with consideration of past trends in requirements, the load-forecast curve is constructed. We have constructed trend curves to show the past power use and the future power requirements for the approximate region mentioned by Mr. Lindner, which is Power Supply Areas 21, 22, 23, and 24, as established by the Federal Power Commission. These curves, consisting of the power components of capacity and energy, are shown on the attached Plate 1. The trend lines on the white background correspond to actual records of power use up to 1955. Those on the dark background represent the estimated future power requirements. It will be noted that the demand for power more than doubled between 1945 and 1952. The maximum load went from 3,224,000 kilowatts to 6,717,000 kilowatts, and the annual energy requirements increased from 16,929,000,000 kilowatt-hours to 34,124,000,000 kilowatt-hours.

Also, this graph indicates that by the end of 1957 the maximum demand will be of the order of 10,200,000 kilowatts, which more than triples the 1945 load. The supply under construction and definitely planned will increase the installed capacity in this region to about 10,750,000 kilowatts by the end of 1957. That amount is more than the 10,585,000 kilowatts of capacity shown in Table 1 of Messrs. Lindner's and Johnson's paper, because the geographic areas for which the capacities have been summarized are not exactly the same. On this basis, if the estimates of future power requirements are proven approximately correct and the additional new supply becomes available as planned, no power shortages in the area will occur prior to 1962. By 1962, however, the maximum demand is estimated to be about 13,900,000

kilowatts, or about double the 1952 load. This amounts to an expansion of nearly 740,000 kilowatts per year from 1957 through 1962, or a total of 3,700,000 kilowatts in the five years.

The question now arises, how will the future power supply, which can be provided by the scheduled and planned Federal projects mentioned by Mr. Lindner, fit this expanded load? We can illustrate the analytical solution of this problem by loading the capability and energy from these projects into the estimated load-duration curve for the peak month of December 1962. This was accomplished by relating the estimated power supply to the forecast of requirements and assuming full hydraulic and electrical coordination within each Power Supply Area. On this assumption for the region under consideration the supply from any source could be made available to any load either by direct transmission or by displacement.

The allocation of the capability and energy expected to be available from the power supply in December 1962 is shown loaded into the expanded load-duration curve for that month on the attached Plate 2. The legend on the figure indicates the type of supply in each loaded block. The steam-electric supply existing and under construction is shown by the light diagonal hatching; the existing hydroelectric supply is indicated by the light vertical hatching; the hydroelectric supply under construction and definitely planned is represented by the heavy diagonal hatching; and the blank band shows the unsatisfied requirements, or the additional supply needed to meet the estimated load in December 1962.

In filling this load-duration curve, full use was made of all generating capacity to the extent of its capability. The allocation of both the capacity and energy components was made in a manner to assure the most appropriate and effective utilization of the supply. The supply from the existing hydroelectric projects and the projects under construction and definitely planned was first loaded on the curve, as symbolized by the light vertically hatched and heavy diagonally hatched areas. Allocation of this supply to the several positions indicated on the curve was determined through consideration for the probable load factor of operation of individual projects or groups of projects in the future load. The steam-electric supply existing and under construction or planned was loaded after the hydroelectric supply. The steam-electric supply is symbolized by the light diagonally hatched areas. The heavy diagonally hatched areas then indicate where the hydro supply under construction and definitely planned must necessarily operate to make the capacity effective and to assure that no duplicating capacity would be provided. You will note that even after loading all the expected future supply from hydro projects under construction and definitely planned, an unfilled band still exists at the extreme top of the load curve consisting of 400,000 kilowatts. Therefore, from this analysis, we find that the power from all the planned and definitely scheduled hydroelectric projects mentioned by Mr. Lindner can be used effectively in the loads of the area under consideration by the latter part of 1962.

In the foregoing analysis of the future supply and its utilization, no capacity was allocated to reserve. If we assume a reserve supply amounting to only five percent of the installed capacity, which most power experts would consider inadequate, the additional requirements for this purpose will amount to about 700,000 kilowatts. On this basis, we may safely conclude that all the hydroelectric power which is likely to be developed in this area during the next 10 years can be absorbed in the expanding loads of the power systems.

Returning now to the discussion of another point in the paper presented by Mr. Lindner, the writer would like to comment on the following statement concerning hydroelectric power sites: "Yet the thought still persists that the remaining water powers are uneconomical. Beyond a doubt this idea has been a large factor in delaying development of our water-power resources." We believe that other factors were more influential in retarding development of hydroelectric power, such as lack of load of the type which could absorb the peaking capacity provided by major hydroelectric projects, and the fact that the steam-electric-hydroelectric ratio has been economically favorable until recently.

PLATE 1
POWER REQUIREMENT TRENDS
POWER SUPPLY AREAS
21, 22, 23, and 24

ESTIMATED .....

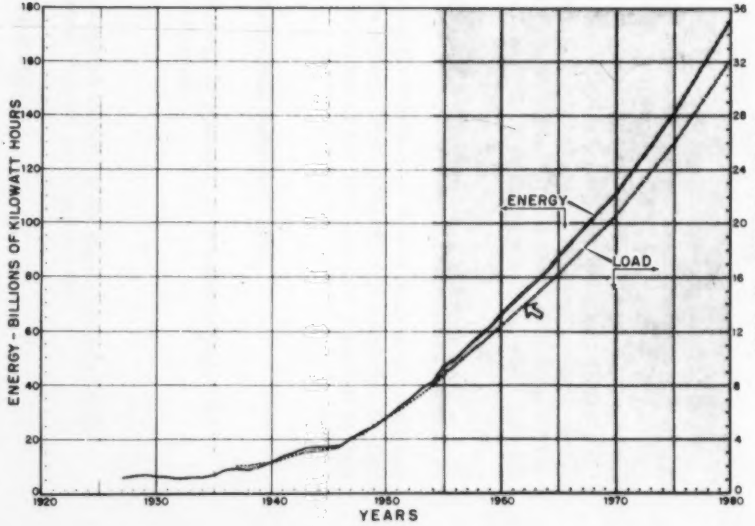
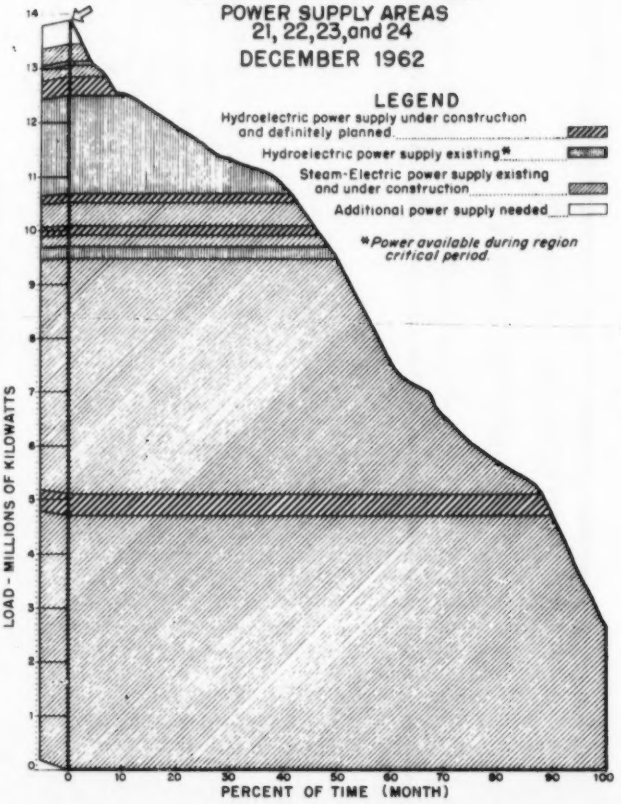


PLATE 2
REPRESENTATIVE LOAD DURATION CURVE
POWER SUPPLY AREAS
21, 22, 23, and 24
DECEMBER 1962


LEGEND
Hydroelectric power supply under construction and definitely planned
Hydroelectric power supply existing*
Steam-Electric power supply existing and under construction
Additional power supply needed
*Power available during region critical period.
PERCENT OF TIME (MONTH)

DIVISION ACTIVITIES
POWER DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

April, 1957

Mr. R. R. Randolph, Southern Service Company, Birmingham, Alabama is now Secretary of the Power Division Executive Committee.

Personnel of the new Committee on Nuclear Energy is now complete with the following members: W. A. Conwell, Chairman, Duquesne Light Company, Pittsburgh, Penn.; M. H. Cutler, Stone & Webster Engineering Corp., Boston, Mass.; F. F. Mautz, Pacific Gas & Electric Company, San Francisco, Calif.; A. G. Villepique, Ebasco Services, Inc., New York, N.Y.; E. S. Harrison, Georgia Power Company, Atlanta, Ga.

Buffalo Convention June 3-7, 1957

A Symposium on Penstocks will be presented consisting of eight papers. This will represent a broad field of practice in penstock design and construction, and will include a paper by R. Bouchayer and S. Ferrand of Grenoble, France. Two other papers will complete the Power Division program which will be presented in four sessions. Field trips will be arranged to the Huntley Steam Station, the Sir Adam Beck hydroelectric development, and an all-day trip to the St. Lawrence development. The charge for the latter including Pullman travel from Buffalo both ways at night will be about $30.00.

Annual Convention, New York October 1957

The Symposium on Underground Hydroelectric Power Plants is now practically complete, and will include papers from a number of countries in addition to the United States. Four sessions of the Division are expected at this meeting.

Chicago Convention February 1958

The Committee on Nuclear Energy is planning to present a paper or papers, and field trips may be arranged.

Note: No. 1957-10 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, PO 2, April, 1957.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

VOLUME 82 (1956)

APRIL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)[c], 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941(SM2), 942(SM2)[c], 943(EM2), 944(EM2), 945(EM2), 946(EM2)[c], 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)[c], 953(HY2), 954(HY2), 955(HY2)[c], 956(HY2), 957(HY2), 958(SA2), 959(PO2), 960(PO2).

MAY: 961(IR2), 962(IR2), 963(CP2), 964(CP2), 965(WW3), 966(WW3), 967(WW3), 968(WW3), 969(WW3), 970(ST3), 971(ST3), 972(ST3)[c], 973(ST3), 974(ST3), 975(WW3), 976(WW3), 977(IR2), 978(AT2), 979(AT2), 980(AT2), 981(IR2), 982(IR2)[c], 983(HW2), 984(HW2), 985(HW2)[c], 986(ST3), 987(AT2), 988(CP2), 989(AT2).

JUNE: 990(PO3), 991(PO3), 992(PO3), 993(PO3), 994(PO3), 995(PO3), 996(PO3), 997(PO3), 998(SA3), 999(SA3), 1000(SA3), 1001(SA3), 1002(SA3), 1003(SA3)[c], 1004(HY3), 1005(HY3), 1006(HY3), 1007(HY3), 1008(HY3), 1009(HY3), 1010(HY3)[c], 1011(PO3)[c], 1012(SA3), 1013(SA3), 1014(SA3), 1015(HY3), 1016(SA3), 1017(PO3), 1018(PO3).

JULY: 1019(ST4), 1020(ST4), 1021(ST4), 1022(ST4), 1023(ST4), 1024(ST4)[c], 1025(SM3), 1026(SM3), 1027(SM3), 1028(SM3)[c], 1029(EM3), 1030(EM3), 1031(EM3), 1032(EM3), 1033(EM3)[c].

AUGUST: 1034(HY4), 1035(HY4), 1036(HY4), 1037(HY4), 1038(HY4), 1039(HY4), 1040(HY4), 1041(HY4)[c], 1042(PO4), 1043(PO4), 1044(PO4), 1045(PO4), 1046(PO4)[c], 1047(SA4), 1048(SA4)[c], 1049(SA4), 1050(SA4), 1051(SA4). 1052(HY4), 1053(SA4).

SEPTEMBER: 1054(ST5), 1055(ST5), 1056(ST5), 1057(ST5), 1058(ST5), 1059(WW4), 1060(WW4), 1061(WW4), 1062(WW4), 1063(WW4), 1064(SU2), 1065(SU2), 1066(SU2)[c], 1067(ST5)[c], 1068(WW4)[c], 1069(WW4).

OCTOBER: 1070(EM4), 1071(EM4), 1072(EM4), 1073(EM4), 1074(HW3), 1075(HW3), 1076(HW3), 1077(HY5), 1078(SA5), 1079(SM4), 1080(SM4), 1081(SM4), 1082(HY5), 1083(SA5), 1084(SA5), 1085(SA5), 1086(PO5), 1087(SA5), 1088(SA5), 1089(SA5), 1090(HW3), 1091(EM4)[c], 1092(HY5)[c], 1093(HW3)[c], 1094(PO5)[c], 1095(SM4)[c].

NOVEMBER: 1096(ST6), 1097(ST6), 1098(ST6), 1099(ST6), 1100(ST6), 1101(ST6), 1102(IR3), 1103(IR3), 1104(IR3), 1105(IR3), 1106(ST6), 1107(ST6), 1108(ST6), 1109(AT3), 1110(AT3)[c], 1111(IR3)[c], 1112(ST6)[c].

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127(SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

VOLUME 83 (1957)

JANUARY: 1136(CP1), 1137(CP1), 1138(EM1), 1139(EM1), 1140(EM1), 1141(EM1), 1142(SM1), 1143(SM1), 1144(SM1), 1145(SM1), 1146(ST1), 1147(ST1), 1148(ST1), 1149(ST1), 1150(ST1), 1151(ST1), 1152(CP1)[c], 1153(HW1), 1154(EM1)[c], 1155(SM1)[c], 1156(ST1)[c], 1157(EM1), 1158(EM1), 1159(SM1), 1160(SM1), 1161(SM1).

FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

MARCH: 1186(ST2), 1187(ST2), 1188(ST2), 1189(ST2), 1190(ST2), 1191(ST2), 1192(ST2)[c], 1193(PL1), 1194(PL1), 1195(PL1).

APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203(SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218(SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226(WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(HY2)[c].

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c. Discussion of several papers, grouped by Divisions.

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